AUGUST 1961

Journal

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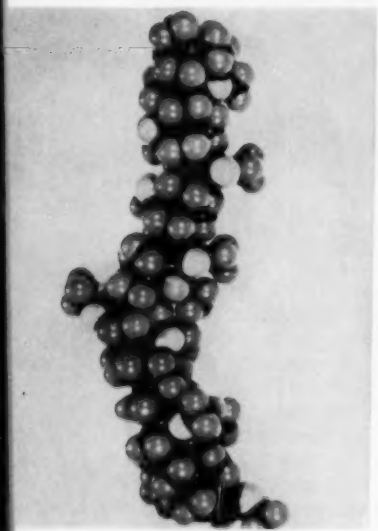
AMERICAN WATER WORKS ASSOCIATION



Long-chain polymer—a building block for better floc

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Committee Report

ON GUARD IN INDEPENDENCE SQUARE

About 75 feet away from Independence Hall, in the park to the south, stand R. D. Wood Hydrants like sentinels on duty. While their proximity to the Nation's most revered historic monument does not necessarily imply endorsement, we nevertheless like to think of them there, on guard night and day against the fire that could destroy what Gen. Howe saw fit to spare, and Clinton too, in the British occupation. But R. D. Wood Hydrants need no endorsement. For over a century they have shown themselves to be in constant readiness for any emergency. Of simple, ingenious design and rugged construction, R. D. Wood Hydrants are protected against ice and sediment—function smoothly when the engines arrive. There could be no better protection for any community than R. D. Wood Hydrants—and there could be no better protection for the birthplace of the U.S.A.

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Manufacturers of "Sand-Spun" Pipe (centrifugally cast in sand molds)

Journal

AMERICAN WATER WORKS ASSOCIATION

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August 1961

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Corrosion resistant coatings, films and foams.

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Plastic fittings, valves and special fabrications.

LJ

LOCK JOINT PIPE COMPANY
Established 1905
P. O. BOX 269, EAST ORANGE, N. J.

Member of The American Concrete Pipe Association and The American Concrete Pressure Pipe Association

CONCRETE PRESSURE, SEWER, CULVERT & SUBAQUEOUS PIPE. PLASTIC PIPE AND PROTECTIVE COATINGS

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POSITIVE CONTROL OF MATERIALS FLOW

MEET PEAK LOAD DEMANDS... WITHOUT EXPENSIVE EXPANSION!

B-I-F Vacuum Filters Provide LOW COST "BUILDING BLOCK" FILTRATION SYSTEM!

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Coming Meetings

AWWA SECTIONS

Fall 1961

Sep. 11–13—Kentucky-Tennessee Section, at Brown Hotel, Louisville, Ky. Secretary, Harold F. Mount, Gen. Mgr., Preston Street Road Water Dist. No. 1, 5400 Preston Hwy., Louisville, Ky.

Sep. 13–14—New York Section, at Saranac Inn, Saranac Lake. Secretary, Kimball Blanchard, Neptune Meter Co., 2222 Jackson Ave., Long Island City 1.

Sep. 13–15—North Central Section, at Pick-Nicollet Hotel, Minneapolis, Minn. Secretary, Carl A. Flack, Registrar, Water Dept., 216 City Hall, St. Paul 2, Minn.

Sep. 20–22—South Dakota Section, at Sheraton-Johnson Hotel, Rapid City. Secretary, J. Darrel Bakken, Div. of San. Eng., State Dept. of Health, Pierre.

Sep. 27–29—Wisconsin Section, at Hotel Schroeder, Milwaukee. Secretary, Harry Breimeister, Bureau of Engineers, Room 607 Municipal Bldg., 841 N. Broadway, Milwaukee.

Sep. 28—Connecticut Section. Secretary, D. W. Loiselle, Supt. of Supply, Bridgeport Hydraulic Co., 835 Main St., Bridgeport.

Oct. 1–3—Missouri Section, at Kentwood Arms Hotel, Springfield. Secretary, Warren A. Kramer, Chief, Water Supply, Div. of Health, State Office Bldg., Jefferson City.

Oct. 2–3—Maritime Branch, Canadian Section, at Brunswick Hotel, Moncton, N.B. Secretary, J. D. Kline, Gen. Mgr., Public Service Com., Halifax, N.S.

Oct. 2–4—Rocky Mountain Section, at La Katchina Hotel, Taos, N.M. Secretary, H. F. Kepner, Vice-Pres., Dana Kepner Co., 550 Alcott, Denver, Colo.

Oct. 4–6—Virginia Section, at Hotel Roanoke, Roanoke. Secretary, Edward H. Ruehl, R. Stuart Royer & Assocs., 15 W. Cary St., Richmond 20.

Oct. 5–6—Intermountain Section, at Twin Falls, Idaho. Secretary, M. W. Snell, Supt., Power & Water, Soda Springs, Idaho.

Oct. 8–11—Alabama-Mississippi Section, at Buena Vista Hotel, Biloxi, Miss. Secretary, Ernest Bryan, Southern Sales Mgr., McWane Cast Iron Pipe Co., Box 2601, Birmingham, Ala.

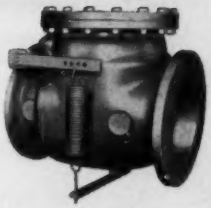
Oct. 15–18—Southwest Section, at Granada Hotel, San Antonio, Tex. Secretary, L. A. Jackson, Mgr.-Engr., Municipal Water Works, Robinson Memorial Auditorium, Little Rock, Ark.

(Continued on page 8)

EDDY New Improved Products Meet Modern Needs

EDDY Fire Hydrant

- Improved, streamlined, modernized
- Optional break-flange
- Large diameter barrel
- New self-sealing main valve packing
- Requires no lubrication
- Opens *with* pressure
- Closes *against* pressure
- Underwriters' listed
- Standardized, interchangeable parts

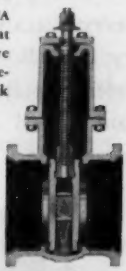


EDDY Check Valves

Completely new designs of horizontal swing check valves are available in sizes from 4 through 12 inches. They operate dependably in either vertical or horizontal positions and incorporate construction features which provide unusual flexibility of operation. These valves may be converted in the field from plain to single- or double-lever operation to meet changing requirements. Other check valves up to 24 inches.

EDDY Gate Valves

In addition to the regular line of EDDY Taper Seat AWWA gate valves, EDDY now offers a newly designed parallel seat AWWA gate valve in sizes through 12 inches. This valve has been specifically engineered to provide long, trouble-free service. Working parts are designed with heavy, thick section and large seating surfaces for dependable operation and improved wearing qualities. The valve has two-point, free-floating wedging for minimum friction and maximum operating ease. Other AWWA gate valves are available through 48 inches.



Whatever your waterworks requirements, it will pay you to have full information on the complete EDDY *line of valves and hydrants.*

EDDY VALVE COMPANY
A Subsidiary of James B. Clow & Sons, Inc.
WATERFORD NEW YORK

Coming Meetings

(Continued from page 6)

Oct. 18–20—Iowa Section, at Roosevelt Hotel, Cedar Rapids. Secretary, Wilbur E. Bjork, Service Supt., Des Moines Water Works, 1003 Locust St., Des Moines.

Oct. 25–27—California Section, at Hotel Senator, Sacramento. Secretary, Frank F. Watters, Hydr. Engr., State Public Utilities Com., State Bldg., Civic Center, San Francisco.

Oct. 25–27—Ohio Section, at Commodore Perry Hotel, Toledo. Secretary, J. H. Bass, Henry P. Thompson Co., 4866 Cooper Rd., Cincinnati.

Oct. 25–28—New Jersey Section, at Dennis Hotel, Atlantic City. Secretary, A. F. Pleibel, District Sales Mgr., R. D. Wood Co., 683 Prospect St., Maplewood.

Oct. 29–Nov. 1—Florida Section, at Cherry Plaza Hotel, Orlando. Secretary, John B. Miller, Director, Div. of Water Supply, Bureau of San. Eng., State Board of Health, Box 210, Jacksonville 1.

Nov. 1–3—Chesapeake Section, at Emerson Hotel, Baltimore, Md. Secretary, Carl J. Lauter, 6955—33rd St. N.W., Washington, D.C.

Nov. 13–15—North Carolina Section at George Vanderbilt Hotel, Asheville. Secretary, T. Z. Osborne, Asst. Director of Public Works, Greensboro.

OTHER ORGANIZATIONS

Sep. 17–21—NEWWA, Poland Springs, Me.

Oct. 4–6—North Dakota Water & Sewage Conference, Gladstone Hotel, Jamestown, N.D.

Oct. 8–12—WPCF, Milwaukee, Wis.

Oct. 16–20—ASCE, Statler-Hilton Hotel, New York, N.Y.

Oct. 17–20—Eastern Water Co. Conference, Atlantic City, N.J.

Oct. 23–28—First Inter-American Congress of Chemical Engineering, San Juan, P.R. Write: Secretary, Institute of Chemical Engineers of Puerto Rico, Box 47, Rio Piedras Sta., P.R.

Nov. 13–17—APHA, Cobo Hall, Detroit, Mich.

Nov. 27–Dec. 1.—28th Exposition of Chemical Industries, Coliseum, New York, N.Y.

SHORT COURSES

Oct. 16–27—Training course on "Plankton Identification and Control," R. A. Taft Sanitary Engineering Center, Cincinnati, Ohio. Write: Chief, Training Program, 4676 Columbia Pkwy., Cincinnati 26, Ohio (or to USPHS regional director).

Oct. 23–27—10th Annual Water Works Management Short Course, sponsored by the Illinois and Indiana sections, AWWA, at Allerton House, Monticello, Ill. Write: Short Course Supervisor, 116B Illini Hall, Champaign, Ill.

Bigger face, better features make new Badger *Easy-Read** a standout

Look at this actual-size Easy-Read face — its digits are ¼" high! This magnetic drive meter has more exclusives: *Sealed, removable register* (comes off without disturbing the meter itself)... *Truly permanent magnetic coupling* (it's rustproof and noncorrosive)... *Interchangeable parts.* And because Easy-Read's register and gear train parts operate in a clean, dry lubricated, sealed housing, fogged registers are completely eliminated.

Ask your Badger representative for full details and demonstration of the new Easy-Read magnetic drive meter.

Split-case Easy-Read meters ⅝" through 2"
Frostproof models ⅝" through 1" x 1¼"

*Pat. applied for in U.S.A. and foreign countries

Badger Meter Mfg. Company

4545 West Brown Deer Road • Milwaukee 23, Wisconsin

Chicago Bridge & Iron Company

Write our nearest office for complete information or estimating figures on steel tanks for municipal service.

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South Pasadena
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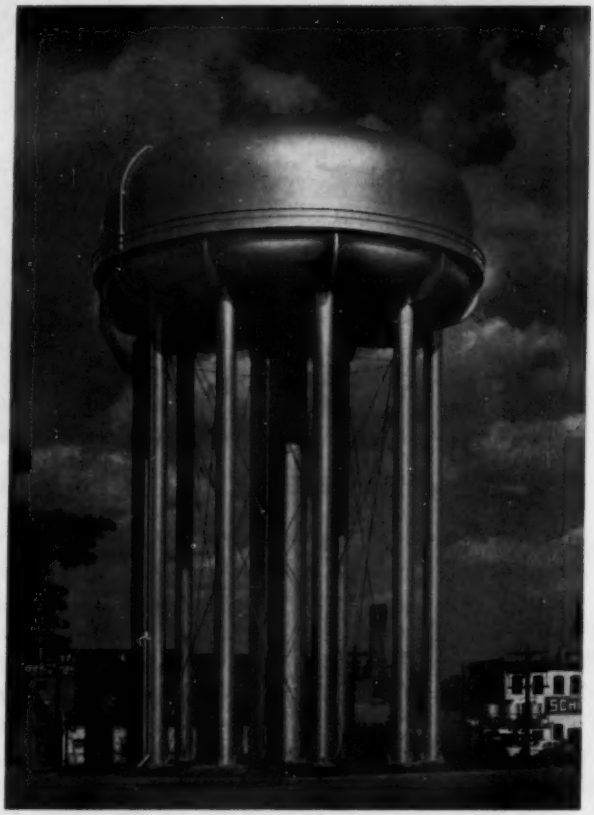
Plants

Birmingham
Chicago
Greenville, Pa.
Houston, Texas
New Castle, Del.
Salt Lake City

IN CANADA:

Horton Steel Works Limited



Cure For A Common Ailment

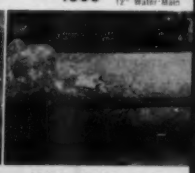
A shortage of water and low water pressure are common problems in many growing communities—but no longer in Mt. Prospect, Illinois. Mt. Prospect's cure came with a thoughtful program of improvement which included construction of a Horton radical cone elevated tank and a steel reservoir. These CB&I-built structures—both of 1,000,000 gallon capacity—helped raise minimum water pressure from 5 to 40 pounds.

PERMANENT

CAST IRON PIPE

1847 BOSTON, MASS. Water Main

1836 LANCASTER, PA. 12" Water Main



1834 READING, PA. 4" Water Main

1830 PHILADELPHIA, PA. 6" Water Main

1858 NEW ORLEANS, LA. 4" Gas Main

1834 POTTSVILLE, PA. Water Main

1831 RICHMOND, VA. 8" Water Main

This advertisement is published in the interests of the Cast Iron Pressure Pipe Industry by . . .

WOODWARD IRON COMPANY

Since 1882

WOODWARD, ALABAMA

B-I-F DALL FLOW TUBES...

The Dall formula . . . for accurately predicting flow characteristics . . . allows great design latitude. With this design flexibility, many styles, many sizes, and the use of a wide variety of construction materials are possible. Whether the installation requires a six-inch insert-type unit of plastic as held by model or a twelve-foot weldment-type of steel, the largest ever made, the distinct Dall Flow Tube advantages are the same:

- Lowest head loss obtainable with any known velocity-increasing differential producer.
- Greatly reduced pumping costs over extended periods . . . adequate line pressure at distant points.
- Self-scouring design ensures enduring accuracy . . . critical dimensions and surfaces kept at optimum condition for years of maintenance-free operation.

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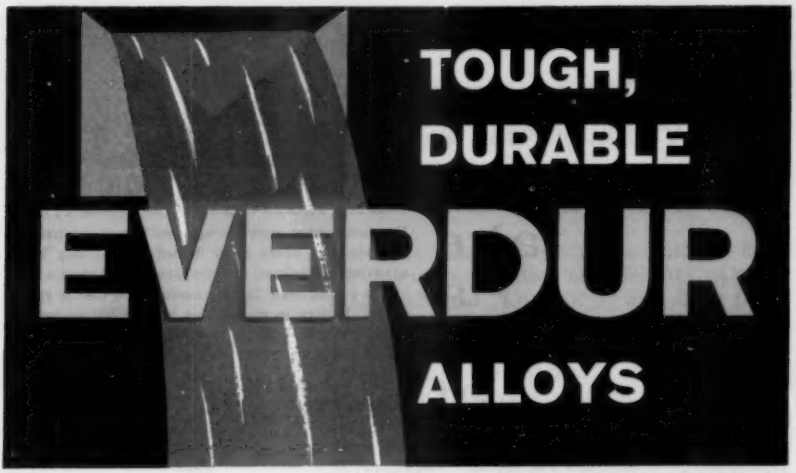

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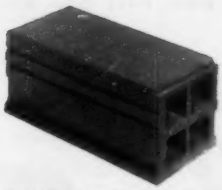

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For more than forty years, Leopold has been serving municipalities with time-tested products, technical "know-how" and filter plant design experience. Now you can specify Leopold for the entire job, whether it be for rebuilding or adding plant capacity. Depend on Leopold's careful handling to produce an effectively engineered operation complete with dependable Leopold products, from "top to bottoms." Write for complete information on any of the products illustrated, to the F. B. Leopold Company, Inc., 227 South Division Street, Zelienople, Pennsylvania.

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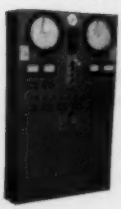

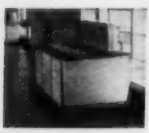

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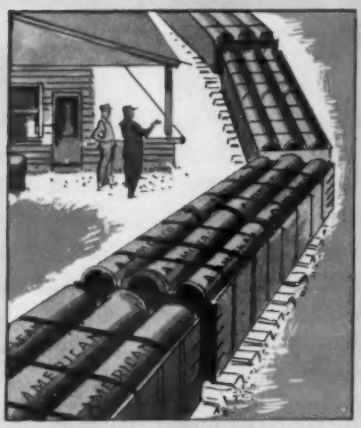
FACTS



DO YOU KNOW that a trainload of 11,000 feet of 36" AMERICAN cast iron pipe recently left Birmingham and arrived in Utica, New York . . . with only one pipe damaged? Such reports are not unusual and prove that cast iron pipe has the strength to withstand shocks and strains in transit . . . and also indicate the care AMERICAN exercises in banding and bracing pipe for shipment.

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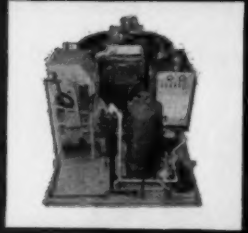

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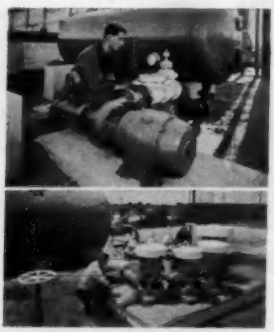
"These Peerless pumps deliver practically all of the 18 million gallons of water we distribute daily."

Says Thomas J. Clemmer, Operations Manager, Dominguez Water Corporation, Long Beach, Calif.



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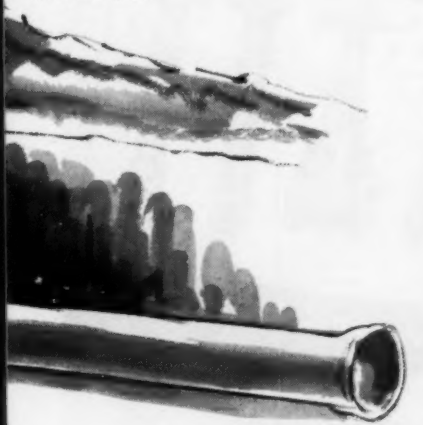

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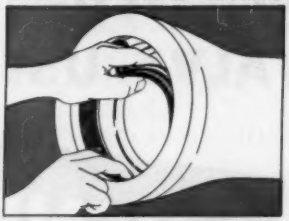
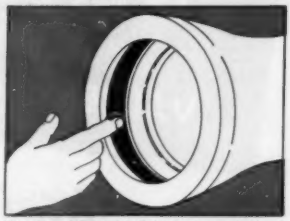
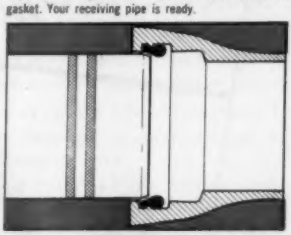


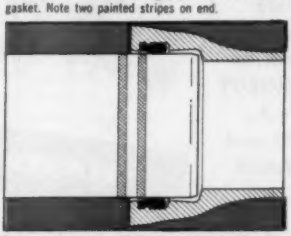
Insert gasket with groove over bead in gasket seat . . . a simple hand operation.



Wipe film of Tyton Joint® lubricant over inside of gasket. Your receiving pipe is ready.



Insert plain end of entering pipe until it touches gasket. Note two painted stripes on end.



Push entering pipe until the first painted stripe disappears and the second stripe is approximately flush with bell face. The joint is sealed . . . bottle-tight, permanently! The job's done . . . . fast, efficiently, economically. Could anything be simpler?

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For complete details write for Bulletin 58

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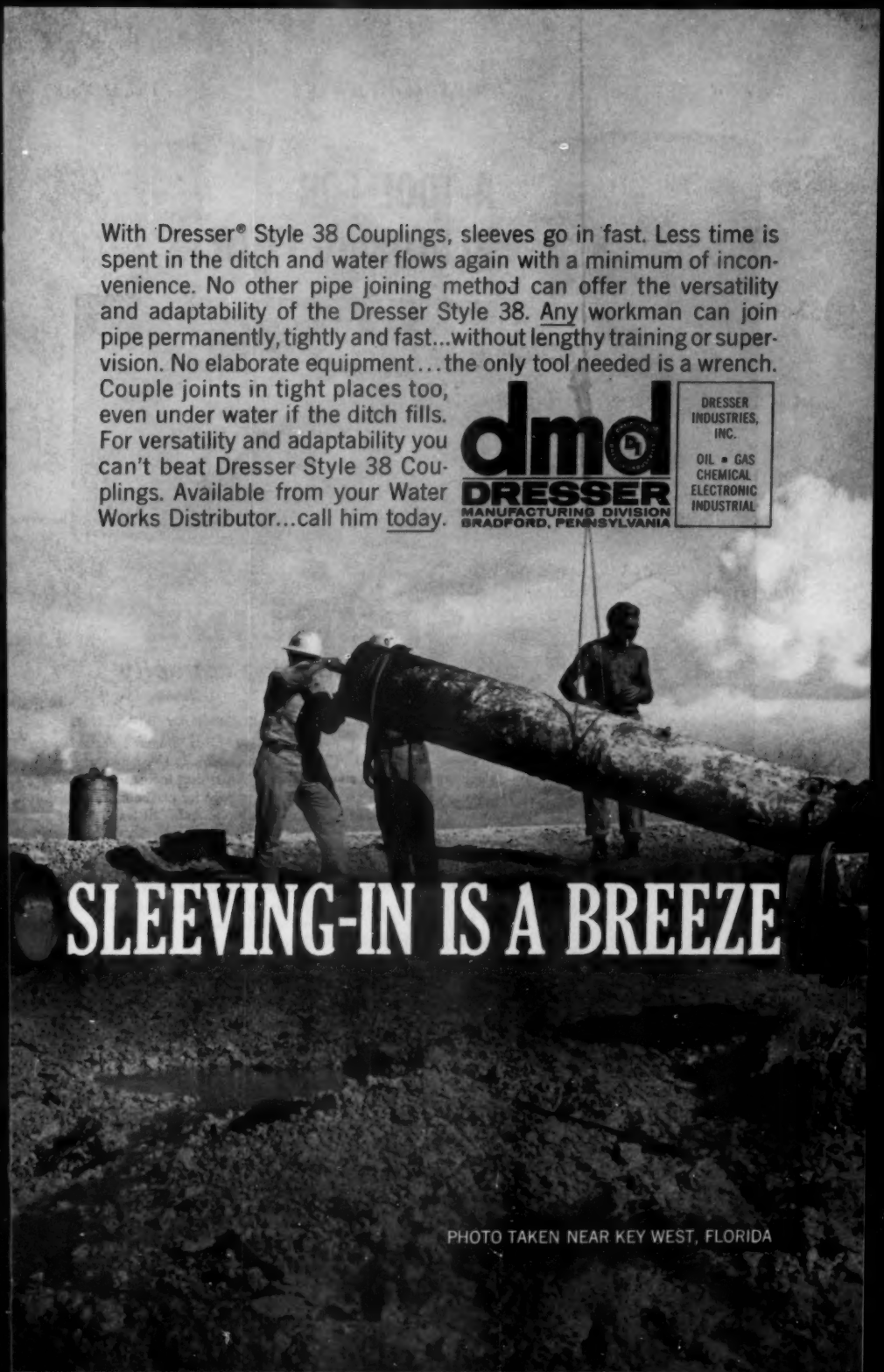
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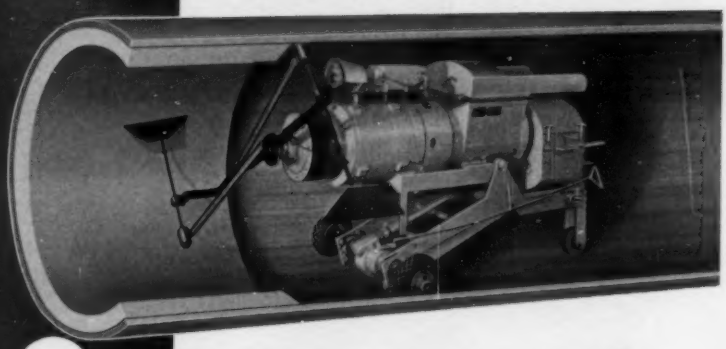
PHOTO TAKEN NEAR KEY WEST, FLORIDA

A TOOL FOR RESTORING FLOW



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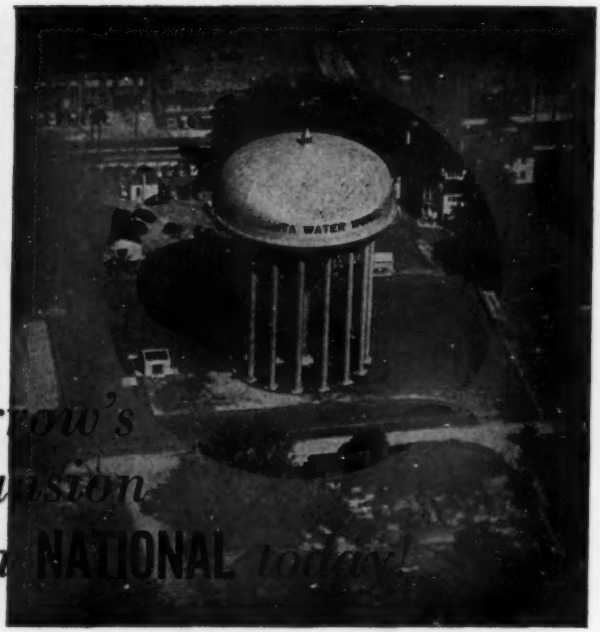

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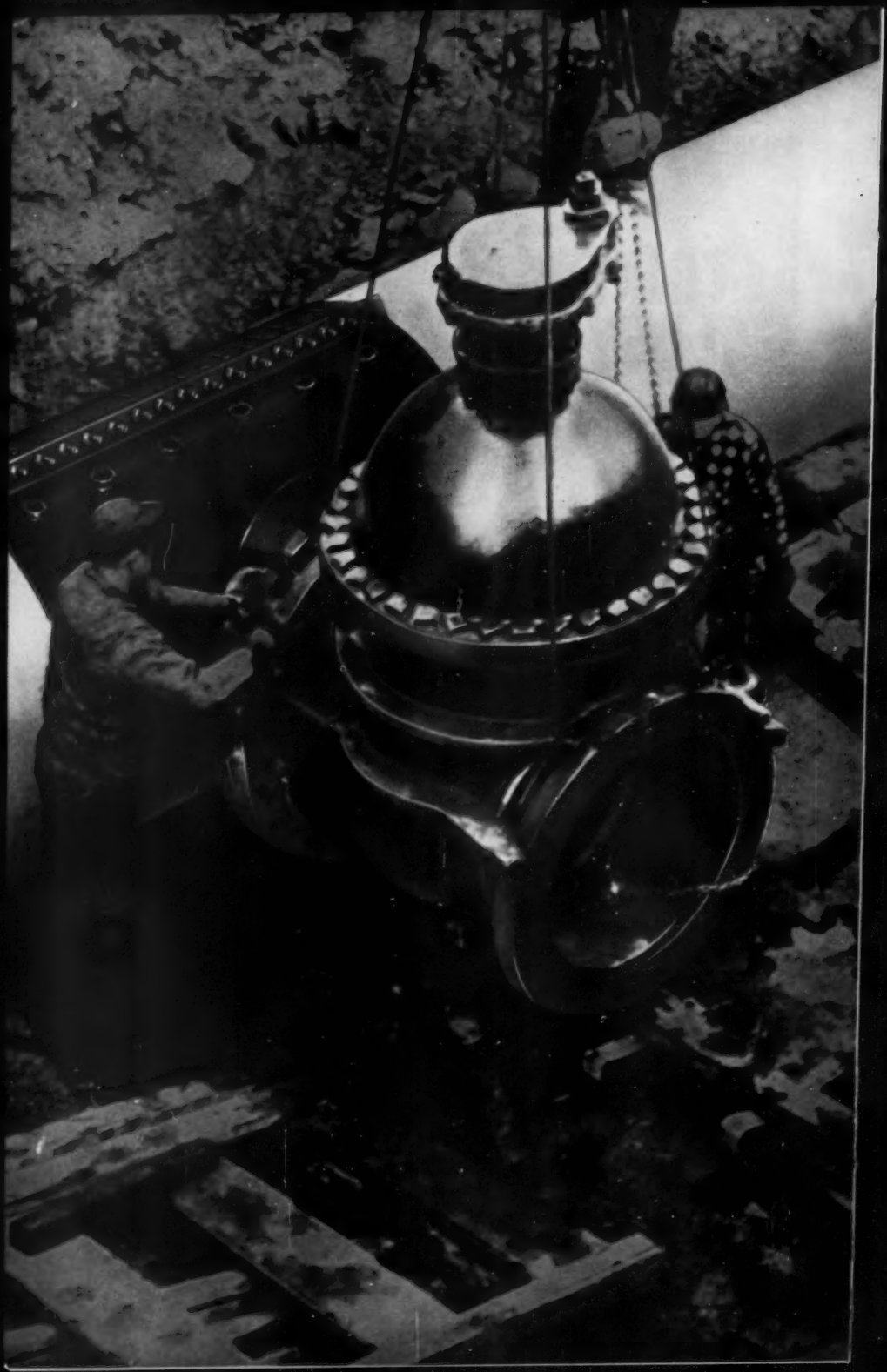

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Journal

AMERICAN WATER WORKS ASSOCIATION

VOL. 53 • AUGUST 1961 • NO. 8

Drinking Water Standards, 1961

US Public Health Service

A report of the Advisory Committee on Revision of USPHS 1946 Drinking Water Standards. The revised standards included in this report have been approved by the Secretary of Health, Education, and Welfare and published in the Federal Register *as the first step in the final approval routine.*

Preface by Advisory Committee

DOMESTIC water supplies should protect the health and promote the well-being of individuals and the community. In this report on the revision of the 1946 edition of the USPHS drinking water standards, the objective of the committee is to recommend minimum requirements for reaching these goals.

The USPHS drinking water standards were first adopted in 1914 to protect the health of the traveling public. The general and widespread use of these standards since that time has led to a series of revisions which have been applicable to water supplies generally. The development of atomic energy and other technological advances require that these standards again be revised. To carry out this revision, the chief sanitary engineer of USPHS appointed the undersigned advisory committee. A technical subcommittee of USPHS officers and a toxicological task force were established to collect information and prepare suggestions for the consideration of the advisory committee.

In preparing this report on the revision of the standards, the committee established the following guidelines:

1. The proposed standards should be discussed widely, and due cognizance should be given to international and other standards of water quality before a final report is submitted.

2. A new section on radioactivity should be added.

3. Greater attention should be given to the chemical substances being encountered increasingly, in both variety and quantity, in water sources.

4. In establishing limits for toxic substances, intake from food and air should be considered.

5. The rationale employed in determining the various limits should be included in an appendix.

6. The proposed format, with the exceptions noted above, should not differ greatly from the present standards.

7. The standards should be generally acceptable and should be applicable to all public water supplies in the United States, as well as to those supplies used by carriers subject to USPHS regulations.

8. The following two types of limits used in previous editions should be continued:

a. Limits which, if exceeded, shall be grounds for rejection of the supply. Substances in this category may have adverse effects on health when present in concentrations above the limit.

b. Limits which should not be exceeded whenever more suitable supplies are, or can be made, available at reasonable cost. Substances in this category, when present in concentrations above the limit, are either objectionable to an appreciable number of people or exceed the levels required by good water quality control practices.

9. These limits should apply to the water at the free-flowing outlet of the ultimate consumer.

This revision of the drinking water standards includes, for the first time, limiting concentrations of radioactivity in water. The effects on large population groups of chronic exposure to low levels of radioactivity are not yet well defined. The limits presented herein are an effort to derive conservative values from the best information now available and may be adjusted upward or downward as new and better data become available.

The committee has taken cognizance of the growing problem of potentially harmful chemicals in sources of drinking water. Limits for several new chemicals have been added, including a gross limit for the concentration of some types of synthetic chemicals. It was not feasible, however, to include limits for all the many chemicals that have varying degrees of toxic potential. Consideration was given to the more common chlorinated hydrocarbon and organophosphate insecticides, but the information available was not sufficient to establish specific limits for these chemicals. Moreover, the concentrations of these chemicals, where tested, have been below those which would constitute a known health hazard. The committee believes that pollution of water supplies with such contaminants can become significant and urges that the problem be kept under closer surveillance. Further, the committee recommends that regulatory actions be taken to minimize concentrations of such chemicals in drinking water.

In view of the accelerating pace of new developments affecting water quality, the committee recommends that a mechanism be established for continual appraisal and appropriate revision of the standards. It also recommends that USPHS intensify its continuing studies toward the development of basic information on the relationship of the biological, chemical, physical, and radiological aspects of water quality to health.

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Drinking Water Standards

1. Definition of Terms

The terms used in these standards are as follows:

1.1. *Adequate protection by natural means* involves one or more of the following processes of nature that produce water consistently meeting the requirements of these standards: dilution, storage, sedimentation, sunlight, aeration, and the associated physical and biological processes which tend to accomplish natural purification in surface waters and, in the case of ground waters, the natural purification of water by infiltration through soil and percolation through underlying material and storage below the ground water table.

1.2. *Adequate protection by treatment* is any one or any combination of the controlled processes of coagulation, sedimentation, sorption, filtration, disinfection, or other processes which produce a water consistently meeting the requirements of these standards. This protection also includes processes which are appropriate to the source of supply; works which are of adequate capacity to meet maximum demands without creating health hazards, and which are located, designed, and constructed to eliminate or prevent pollution; and conscientious operation by well trained and competent personnel whose qualifications are commensurate with the responsibilities of the

position and acceptable to the reporting agency and the certifying authority.

1.3. *Certifying authority* is the surgeon general of USPHS or his duly authorized representatives. (Reference to the certifying authority is applicable only for those water supplies to be certified for use on carriers subject to the USPHS regulations—42 CFR Part 72.)

1.4. The *coliform group* includes all organisms considered in the coliform group as set forth in *Standard Methods for the Examination of Water and Wastewater,* current edition [11th ed., 1960], prepared and published jointly by the American Public Health Association, American Water Works Association, and Water Pollution Control Federation.

1.5. *Health hazards* are conditions, devices, or practices in the water supply system and its operation which create, or may create, a danger to the health and well-being of the water consumer. An example of a health hazard is a structural defect in the water supply system, whether of location, design, or construction, which may regularly or occasionally prevent satisfactory purification of the water supply or cause it to be polluted from extraneous sources.

1.6. *Pollution,* as used in these standards, is defined as the presence of any foreign substance (organic, inorganic, radiological, or biological) in water which tends to degrade its quality so as to constitute a hazard or impair the usefulness of the water.

1.7. *Reporting agencies* are the respective official state health agencies or their designated representatives.

1.8. *The standard sample* for the bacteriological test shall consist of:

1.81. For the bacteriological fermentation tube test, five standard portions of either: (a) 10 ml or (b) 100 ml

1.82. For the membrane filter technique, not less than 50 ml.

1.9. *Water supply system* includes the works and auxiliaries for collection, treatment, storage, and distribution of the water from the sources of supply to the free-flowing outlet of the ultimate consumer.

2. Source and Protection

2.1. The water supply should be obtained from the most desirable source feasible, and effort should be made to prevent or control pollution of the source. If the source is not adequately protected by natural means, the supply shall be adequately protected by treatment.

2.2. Frequent sanitary surveys shall be made of the water supply system to locate and identify health hazards which might exist in the system. The manner and frequency of making these surveys, and the rate at which discovered health hazards are to be removed, shall be in accordance with a program approved by the reporting agency and the certifying authority.

2.3. Approval of water supplies shall be dependent in part upon:

a. Enforcement of rules and regulations to prevent development of health hazards

b. Adequate protection of the water quality throughout all parts of the system, as demonstrated by frequent surveys

c. Proper operation of the water supply system under the responsible charge of personnel whose qualifications are acceptable to the reporting agency and the certifying authority

d. Adequate capacity to meet peak demands without development of low pressures or other health hazards

e. Record of laboratory examinations showing consistent compliance with the water quality requirements of these standards.

2.4. For the purpose of application of these standards, responsibility for the conditions in the water supply system shall be considered to be held by:

a. The water purveyor, from the source of supply to the connection to the customer's service piping

b. The owner of the property served and the municipal, county, or other authority having legal jurisdiction, from the point of connection to the customer's service piping to the free-flowing outlet of the ultimate consumer.

3. Bacteriological Quality

3.1. *Sampling.*

3.11. Compliance with the bacteriological requirements of these standards shall be based on examinations of samples collected at representative points throughout the distribution system. The frequency of sampling and the location of sampling points shall be established jointly by the reporting agency and the certifying authority after investigation by either agency, or both, of the source, method of treatment, and protection of the water concerned.

3.12. The minimum number of samples to be collected from the distribution system and examined each month should be in accordance with the number in Fig. 1 for the population served by the system. For the purpose of uniformity and simplicity in application, the number determined from Fig. 1 should be in accordance with the following: for a population of 25,000 or less, to the nearest 1; 25,001–100,000, to the nearest 5; and more than 100,000, to the nearest 10.

3.13. In determining the number of samples examined monthly, the following samples may be included, provided all results are assembled and available for inspection, and the laboratory methods and technical competence of the laboratory personnel are approved by the reporting agency and the certifying authority:

a. Samples examined by the reporting agency

b. Samples examined by local government laboratories

c. Samples examined by the water works authority

d. Samples examined by commercial laboratories.

3.14. The laboratories in which these examinations are made and the methods used in making them shall be subject to inspection at any time by the designated representatives of the certifying authority and the reporting agency. Compliance with the specified procedures and the results obtained shall be used as a basis for certification of the supply.

3.15. Daily samples collected following a bacteriologically unsatisfactory sample as provided in Sec. 3.21, 3.22, and 3.23 shall be considered as special samples and shall not be included in the total number of samples examined. Neither shall such special samples be used as a basis for prohibiting the supply, provided that:

1. When waters of unknown quality are being examined, simultaneous tests are made on multiple portions of a geometric series to determine a definitive coliform content

2. Immediate and active efforts are made to locate the cause of pollution

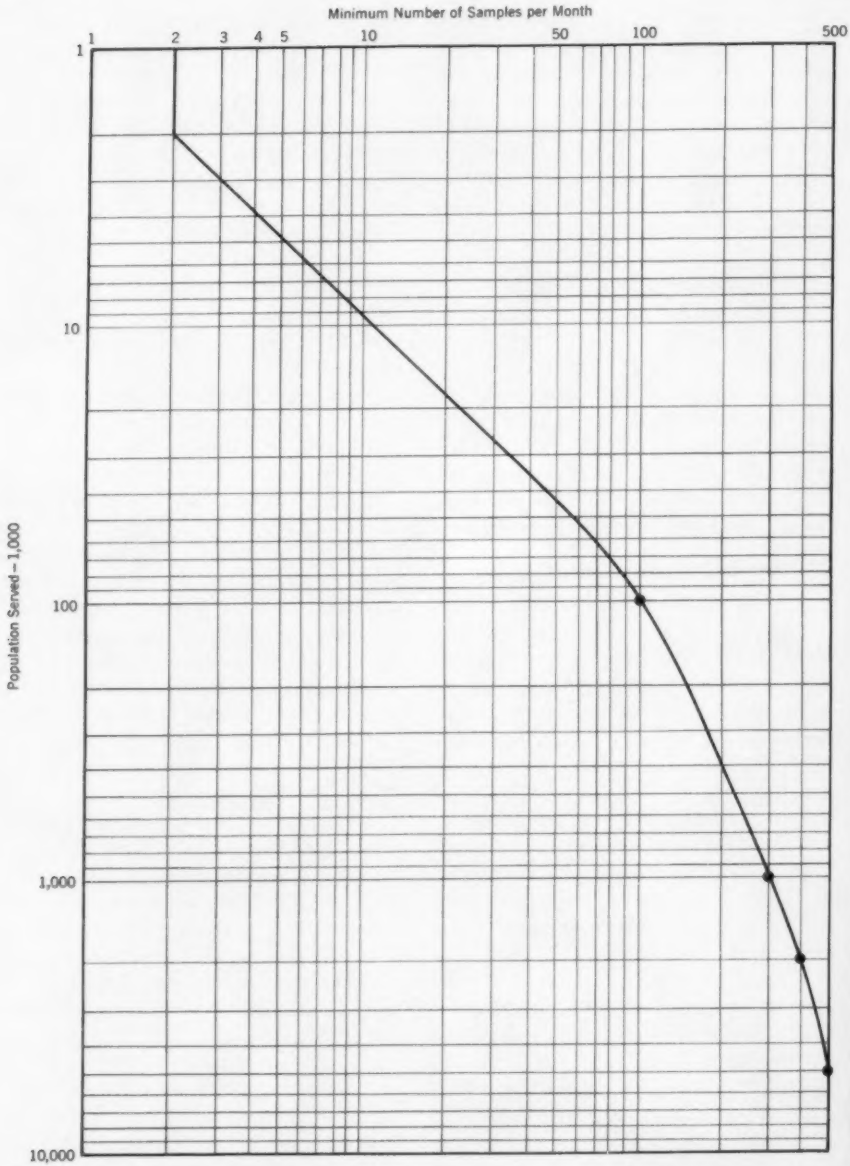

Fig. 1. Relation Between Minimum Number of Samples to Be Collected per Month and Population Served

3. Immediate action is taken to eliminate the cause

4. Samples taken following such remedial action are satisfactory.

3.2. *Limits.* The presence of organisms of the coliform group as indicated by samples examined shall not exceed the following limits:

3.21. When 10-ml standard portions are examined, not more than 10 per cent in any month shall show the presence of the coliform group. The presence of the coliform group in three or more 10-ml portions of a standard sample shall not be allowable if this occurs:

a. In two consecutive samples

b. In more than one sample per month when less than twenty are examined per month

c. In more than 5 per cent of the samples when twenty or more are examined per month.

When organisms of the coliform group occur in three or more of the 10-ml portions of a single standard sample, daily samples from the same sampling point shall be collected promptly and examined until the results obtained from at least two consecutive samples show the water to be of satisfactory quality.

3.22. When 100-ml standard portions are examined, not more than 60 per cent in any month shall show the presence of the coliform group. The presence of the coliform group in all five of the 100-ml portions of a standard sample shall not be allowable if this occurs:

a. In two consecutive samples

b. In more than one sample per month when less than five are examined per month

c. In more than 20 per cent of the samples when five or more are examined per month.

When organisms of the coliform group occur in all five of the 100-ml portions of a single standard sample, daily samples from the same sampling point shall be collected promptly and examined until the results obtained from at least two consecutive samples show the water to be of satisfactory quality.

3.23. When the membrane filter technique is used, the arithmetic mean coliform density of all standard samples examined per month shall not exceed one per 100 ml. Coliform colonies per standard sample shall not exceed three per 50 ml, four per 100 ml, seven per 200 ml, or thirteen per 500 ml in:

a. Two consecutive samples

b. More than one standard sample when less than twenty are examined per month

c. More than 5 per cent of the standard samples when twenty or more are examined per month.

TABLE 1

Refer to Sec. 5.21

Substance	Concentration *mg/l*
Alkyl benzene sulfonate (ABS)	0.5
Arsenic (As)	0.01
Chloride (Cl)	250.0
Copper (Cu)	1.0
Carbon chloroform extract (CCE)	0.2
Cyanide (CN)	0.01
Fluoride (F)	*
Iron (Fe)	0.3
Manganese (Mn)	0.05
Nitrate (NO_3)†	45.0
Phenols	0.001
Sulfate (SO_4)	250.0
Total dissolved solids	500.0
Zinc (Zn)	5.0

* See Sec. 5.23.

† In areas in which the nitrate content of water is known to be in excess of the listed concentration, the public should be warned of the potential dangers of using the water for infant feeding.

When coliform colonies in a single standard sample exceed the above values, daily samples from the same sampling point shall be collected promptly and examined until the results obtained from at least two consecutive samples show the water to be of satisfactory quality.

4. Physical Characteristics

4.1. *Sampling.* The frequency and manner of sampling shall be determined by the reporting agency and the certifying authority. Under normal circumstances, samples should be collected one or more times per week from representative points in the distribution system and examined for turbidity, color, threshold odor, and taste.

4.2. *Limits.* Drinking water should contain no impurity which would cause offense to the sense of sight, taste, or smell. Under general use, the following limits should not be exceeded: turbidity, 5 units; color, 15 units; and threshold odor number, 3 units.

5. Chemical Characteristics

5.1. *Sampling.* The frequency and manner of sampling shall be determined by the reporting agency and the certifying authority. Under normal circumstances, analyses for substances listed below need be made only semiannually. If, however, there is some presumption of unfitness because of the presence of undesirable elements, compounds, or materials, periodic determinations for the suspected toxicant or material should be made more frequently, and an exhaustive sanitary survey should be made to determine the source of the pollution. Where the concentration of a substance is not expected to increase in processing and distribution, available and acceptable source water analyses performed in accordance with standard methods may be used as evidence of compliance with these standards.

Where experience, examination, and available evidence indicate that particular substances are consistently absent from a water supply or below levels of concern, semiannual examinations for those substances may be omitted when approved by the reporting agency and the certifying authority.

The burden of analysis may be reduced in many cases by using data from acceptable sources. Judgment concerning the quality of water supply and the need for performing specific local analyses may depend in part on information produced by such agencies as: (1) USGS, which determines chemical quality of surface and ground waters of the United States and publishes these data in "Water Supply Papers" and other reports, and (2) USPHS, which determines water quality related to pollution (or the absence of pollution) in the principal rivers of the United States and publishes these data annually in "National Water Quality Network." Data on pollution of waters as measured by carbon chloro-

TABLE 2

Refer to Sec. 5.22

Substance	Concentration mg/l
Arsenic (As)	0.05
Barium (Ba)	1.0
Cadmium (Cd)	0.01
Chromium (Cr^{6+})	0.05
Cyanide (CN)	0.2
Fluoride (F)	*
Lead (Pb)	0.05
Selenium (Se)	0.01
Silver (Ag)	0.05

* See Sec. 5.23.

form extracts (CCE) may be found in the latter publication.

5.2. *Limits.* Drinking water shall not contain impurities in concentrations which may be hazardous to the health of the consumers. It should not be excessively corrosive to the water supply system. Substances used in its treatment shall not remain in the water in concentrations greater than required by good practice. Substances which may have deleterious physiological effect, or substances for which physiological effects are not known, shall not be introduced into the system in a manner which would permit them to reach the consumer.

5.21. The chemical substances shown in Table 1 should not be present in a water supply in excess of the listed concentrations where, in the judgment of the reporting agency and the certifying authority, other more suitable supplies are or can be made available.

5.22. The presence of substances in excess of the concentration listed in Table 2 shall constitute grounds for rejection of the supply.

5.23. *Fluoride.* When fluoride is naturally present in drinking water, the concentration should not average more than the appropriate upper limit shown in Table 3. Presence of fluoride in average concentrations greater than two times the optimum values in Table 3 shall constitute grounds for rejection of the supply.

Where fluoridation (supplementation of fluoride in drinking water) is practiced, the average fluoride concentration shall be kept within the upper and lower control limits shown in Table 3.

In addition to the sampling required by Sec. 5.1, fluoridated and defluoridated supplies shall be sampled with sufficient frequency to determine that the desired fluoride concentration is maintained.

6. Radioactivity

6.1. *Sampling.* The frequency of sampling and analysis for radioactivity shall be determined by the reporting agency and the certifying authority after consideration of the likelihood of significant amounts being present. Where concentrations of Ra^{226} or Sr^{90} may vary considerably, quarterly samples composited over a period of 3 months are recommended. Samples for determination of gross activity should be taken and analyzed more frequently.

TABLE 3

Refer to Sec. 5.23

Annual Avg of Max. Daily Air Temperatures* °F	Recommended Control Limits		
	Lower	Optimum	Upper
	Fluoride Concentration—mg/l		
50.0–53.7	0.9	1.2	1.7
53.8–58.3	0.8	1.1	1.5
58.4–63.8	0.8	1.0	1.3
63.9–70.6	0.7	0.9	1.2
70.7–79.2	0.7	0.8	1.0
79.3–90.5	0.6	0.7	0.8

* Based on temperature data obtained for a minimum of 5 years.

As indicated in Sec. 5.1, data from acceptable sources may be used to indicate compliance with these requirements.

6.2. *Limits.* The effects of human radiation exposure are viewed as harmful, and any unnecessary exposure to ionizing radiation should be avoided. The concentrations of radioactivity specified in Table 4 for drink-

ing water are intended to limit intake of the substances by this route, so that total radiation exposure of population groups does not exceed appropriate "Radiation Protection Guides" recommended by the Federal Radiation Council. Concentrations which exceed, on the average, the values presented in Table 4 for a period of 1 year shall constitute grounds for rejection of the supply. Where the total intake of Ra^{226} and Sr^{90} from all sources has been determined, the limits may be adjusted by the reporting agency and the certifying authority so that the total intake of Ra^{226} and Sr^{90} will not exceed 7.3 $\mu\mu c$/day and 73 $\mu\mu c$/ day, respectively.

TABLE 4

Refer to Sec. 6.2

Radionuclides	Concentration $\mu\mu c/l$
Ra^{226}	3
Sr^{90}	10
Gross beta activity (Sr^{90} and alpha emitters absent*)	1,000

* Absent is taken here to mean a negligibly small fraction of the above specific limits, where the limit for unidentified alpha emitters is taken as the listed limit for Ra^{226}.

6.21. When mixtures of Ra^{226}, Sr^{90}, and other radionuclides are present, the above limiting values shall be modified to insure that the combined intake is not likely to result in radiation exposure in excess of the radiation protection guides recommended by the Federal Radiation Council.

7. Recommended Analytical Methods

7.1. Analytical methods to determine compliance with the requirements of these standards shall be those specified in *Standard Methods for the Examination of Water and Wastewater,* APHA, AWWA & WPCF, New York, current edition [11th ed., 1960], and those specified as follows:

7.2. *Barium:* RAINWATER, F. H. & THATCHER, L. L. Methods for the Collection and Analysis of Water Samples. *USGS Wtr. Supply Papers, No. 1454,* Govt. Printing Office, Washington, D.C.

7.3. *Carbon chloroform extract (CCE):* Method for Determining the Carbon Chloroform Extract (CCE) in Drinking Water. R. A. Taft San. Eng. Center, USPHS, Cincinnati (1961).

7.4. *Radioactivity:* Laboratory Manual of Methodology, Radionuclide Analysis of Environmental Samples. Tech. Rept. R59-6, R. A. Taft San. Eng. Center, USPHS, Cincinnati; and Methods of Radiochemical Analysis. Tech. Rept. 173, Joint WHO-FAO Committee, World Health Organization (1959).

7.5. *Selenium:* MAGIN, G. B., ET AL. Suggested Modified Method for Colorimetric Determination of Selenium in Natural Water. *Jour. AWWA,* 52:1199 (Sep. 1960).

7.6. *Organisms of the coliform group:* All of the details of techniques in the determination of bacteria of this group, including the selection and preparation of apparatus and media, the collection and handling of samples, and the intervals and conditions of storage allowable between collection and examination of the water sample, shall be in accordance with *Standard Methods for the Examination of Water and Wastewater,* current edition, and the procedures shall be those specified therein for:

7.61. Membrane Filter Technique, standard test, or

7.62. Completed Test, or

7.63. Confirmed Test, procedure with brilliant green lactose bile broth *

7.64. Confirmed Test, procedure with Endo or eosin methylene blue agar plates.*

* The Confirmed Test is allowed, provided the value of this test to determine the sanitary quality of the specific water supply being examined is established beyond reasonable doubt by comparisons with Completed Tests performed on the same water supply.

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Attitude of US Public Health Service in Development of Drinking Water Standards, 1961

Omar C. Hopkins

A paper presented on Jun. 7, 1961, at the Annual Conference, Detroit, Mich., by Omar C. Hopkins, Deputy Chief, Div. of Water Supply & Pollution Control, USPHS, Washington, D.C.

THE advisory committee on revision of 1946 USPHS drinking water standards has submitted its report to the surgeon general. It is, therefore, appropriate to discuss the attitude of USPHS toward the development and use of these standards. USPHS, of course, has a much broader interest in water than that represented by these standards of drinking water quality. It is vitally concerned with the protection of the quality of *all* water sources for the maximum benefit of all legitimate users. In his special message on natural resources to Congress, President Kennedy stated:

> Pollution of our country's rivers and streams has—as a result of our rapid population and industrial growth and change—reached alarming proportions. To meet all needs—domestic, agricultural, industrial, recreational—we shall have to use and reuse the same water, maintaining quality as well as quantity. In many areas of the country we need new sources of supply—but in all areas we must protect the supplies we have.

If the United States is to have drinking water that meets acceptable standards, the source of supply must be of reasonably good quality. To reuse water as it is today in many places, it is necessary that pollution be kept out of water sources to the maximum extent possible. In simple terms, used water returned to the streams must be as clean as possible.

It is apparent that everyone has a vital interest in the quality of source waters as well as in the quality of drinking water, but this article is concerned primarily with the rationale for the development and use of the present revision of the USPHS drinking water standards.

Drinking Water Standards, 1961

A progress report on the development of the new drinking water standards was presented at the 1960 AWWA Conference at Bal Harbour, Fla.[1] The suggestions received have been incorporated into the advisory committee report, which has been published.

The advisory committee was composed of representatives of thirteen national professional societies, three federal agencies, and one member at large, all directly concerned with the quality of drinking water. The committee members represented broad professional and geographical interests. This representation, plus the high technical competence of the committee members, is assurance that the new standards may be accepted with confidence.

In the preface to its report, the committee explained its aims in the development of the proposed standards:

> Domestic water supplies should protect the health and promote the well-being of individuals and the community. In this report on the revision of the 1946 edition of the USPHS drinking water standards, the objective of the committee is to recommend minimum requirements for reaching these goals.[2]

Domestic water supplies should be more than safe. They should contribute to the health and well-being of the community and of the individual consumers. As these standards are also legal requirements of USPHS and other organizations, they are intended to be minimum requirements. Many people have suggested that these standards should represent higher goals—such as quality standards—so as to provide an incentive to produce the highest quality water possible. Such an approach was not possible under the charge to the committee, as USPHS must have standards that can be upheld in court. USPHS is in sympathy with this objective, however, and hopes that AWWA will develop such standards.

Guidelines for Limits

Of special interest are the following three guidelines used by the committee in establishing limits for impurities in drinking water:

1. Greater attention should be given to the chemical substances encountered increasingly in both variety and quantity in water sources.
2. In establishing limits for toxic substances, the minimum intake from food and air should be considered.
3. Two types of limits used in previous editions should be continued[2]:

a. Limits that, if exceeded, shall be grounds for rejection of the supply. Substances in this category may have adverse effects on health when present in concentrations above the limit.

b. Limits that should not be exceeded whenever more suitable supplies are, or can be made, available at reasonable cost. Substances in this category, when present in concentrations above the limit, are either objectionable to an appreciable number of people or exceed the levels required by good water quality control practices.

It was decided to follow the pattern of earlier standards, which are familiar and which have been well received, except where sound reasons recommended change. A very conservative approach was taken in the proposed revision.

In setting limits for substances in water that may have adverse effects on health when present in concentrations above the limit, intake from air, food, and other sources was considered. For example, it is proposed to reduce the limit for lead from 0.1 ppm to 0.05 ppm because the lead content of air is increasing and the lead in food probably cannot be further reduced. It was also believed that in no event should the limit for a toxic substance exceed that required by the best of modern water quality control practices.

With reference to the limits that should not be exceeded whenever more suitable supplies are or can be available at reasonable cost, the committee had in mind substances such as chlorides or detergents that are not directly injurious to health, but might be objectionable and cause people to use other sources of supply that may not be properly protected. The inclusion

of criteria to insure good practices was also a major objective because poor practice is in itself an inherent health hazard. Again, a rather conservative approach was used. Many people may feel that the limits in this category are too loose rather than too strict.

Chemicals in Water

Considerable attention was given to the growing problem of potentially harmful chemicals in drinking water. Limits for several new chemicals were added, including a gross limit for the concentration of some types of synthetic chemicals. It was not feasible, however, to include specific limits for each of the many chemicals that have varying degrees of toxic potential. For example, many suggestions were made that limits should be set for the chlorinated hydrocarbons and organophosphate insecticides. Even though limits for many insecticides had been set for foods, it was believed that insufficient information was available to set such limits for drinking water. The concentration of these chemicals in water, where tested, has been below that which would constitute a known health hazard. The committee did not want to set the limits so high as to be impracticable, or so low as to encourage further pollution of water. In view of the lack of information on the presence of insecticides in water, and of the absence of known health hazards, no specific action was taken. The committee did state its belief, however, that pollution of water supplies with such contaminants can become significant, and it urged that the problem be kept under surveillance. Further, the committee recommended that regulatory action be taken to minimize the concentrations of such chemicals in drinking water.[2]

USPHS shares the committee's concern about the possible pollution of United States water supply sources by these chemicals and the limited information available concerning the extent of this pollution. Despite limited resources, more data are being gathered on this subject. During the committee discussion of insecticides, it was pointed out that chlorinated hydrocarbons are many times more toxic to fish than to men, and that fish might be used in a screening test for chlorinated hydrocarbons and other insecticides. Unfortunately, the committee did not have information to show that all species of fish are more susceptible than are people to the various organophosphates. More data are needed. It is believed, however, that many fish living in natural water are good indicators of safety, whereas absence of fish or the occurrence of fish kills raises questions of safety that should be investigated. Those concerned with water supply should join with conservation and recreation people in the battle on pollution.

In the concluding paragraph of the preface to its report, the committee stated:

> In view of the accelerating pace of new developments affecting water quality, the committee recommends that a mechanism be established for continual appraisal and appropriate revision of the standards. It also recommends that USPHS intensify its continuing studies toward the development of basic information on the relationship of the biological, chemical, physical, and radiological aspects of water quality to health.[2]

With increasing technological development and rapid urban population growth, the water supply sources will become more polluted. The reuse of the same water several times will be-

come more common. Therefore, everyone concerned with water quality must do everything possible to reduce or eliminate pollution of source waters. In spite of these efforts, however, waters will get worse instead of better. More must be known about the relationship of the biological, chemical, physical, and radiological aspects of water quality to health. And as new knowledge is acquired, people concerned with water quality must be prepared to revise standards and criteria more frequently to meet the changing conditions.

Just as the courts find the reports of congressional committees valuable in interpreting the law, so has the committee that recommended the revisions of drinking water standards found it advisable and helpful to include in an appendix the rationale and supporting references used in determining the proposed requirements. The committee also believed the appendix would serve as an incentive to update the standards as new knowledge is acquired.

Illustrative Requirements

So far the basic attitudes or approaches of the advisory committee and USPHS in developing the new drinking water standards have been discussed. Now a few specific requirements that seem most illustrative will be mentioned.

> The water supply should be obtained from the most desirable source which is feasible, and effort should be made to prevent or control pollution of the source. If the source is not adequately protected by natural means, the supply shall be adequately protected by treatment.[2]

Here the standards recognize the value of good water and the fact that the quality of the raw water often determines the quality of the finished water.

Section 2.3 of the standards is of particular concern to management.

> 2.3. Approval of water supplies shall be dependent in part upon:
>
> *a.* Enforcement of rules and regulations to prevent development of health hazards
>
> *b.* Adequate protection of the water quality throughout all parts of the system, as demonstrated by frequent surveys
>
> *c.* Proper operation of the water supply system under the responsible charge of personnel whose qualifications are acceptable to the reporting agency and the certifying authority
>
> *d.* Adequate capacity to meet peak demands without development of low pressures or other health hazards
>
> *e.* Record of laboratory examinations showing consistent compliance with the water quality requirements of these standards.[2]

Good management and qualified operators are essential for a good product.

Discussion following presentation of the progress report at the 1960 AWWA Conference, Bal Harbour, Fla., indicated the desirability of separating fluorides from the other chemicals. In doing so the committee has recognized three important factors:

1. The amount of water people drink, and hence the fluoride intake, varies with the air temperature
2. Where fluoridation (supplementation of fluoride) is practiced, only the best of modern practice should be tolerated
3. There is an upper limit for fluorides occurring naturally in water which should not be exceeded in drinking water.

The last factor, which illustrates the committee's attitude in developing the new standards, is the section on radioactivity, which is being included for

the first time. The committee included limits for Ra^{226}, Sr^{90} and gross Beta activity (Sr^{90} and Alpha emitter essentially absent), as these radionuclides are of most concern in water supplies. Here again intake from other sources was considered. In the case of Ra^{226}, it appears likely that intake much above the average for the population as a whole usually results from the use of drinking water containing larger than average amounts, as with certain ground waters, or as a result of industrial discharges of wastes containing radium. Therefore, the limit for Ra^{226} in drinking water is relatively high. On the other hand, human intake of Sr^{90} has been primarily from food. Therefore, the limit for Sr^{90} in drinking water is relatively low.

Summary

The advisory committee has completed its task and the new standards are being promulgated. The basic assumptions that guided the development of the new standards may be summarized as follows:

1. Domestic water supplies should protect the health and promote the well-being of individuals and of the community. These proposed standards are minimum requirements for reaching this goal.

2. These minimum requirements should be based in part on compliance with good practices so as not to encourage any unnecessary contamination of drinking water.

3. In many areas it will be necessary to use and reuse the same water, and to maintain quality as well as quantity.

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Relationship Between Banks and the Water Industry

Milton J. Redlich

A paper presented on Jun. 5, 1961, at the Annual Conference, Detroit, Mich., by Milton J. Redlich, Vice-Pres., The Chase Manhattan Bank, New York, N.Y.

OF the many relationships that exist between commercial banks and the various groups that make up the water industry, only a few will be discussed here. One of the most obvious relationships is, of course, commercial banking service, including credit extended to the manufacturers of materials used in building water facilities. Commercial banks also lend money to public and private entities that supply water; and when private corporations sell bonds, banks act as trustees and paying agents.

The Federal Banking Act of 1933 permits commercial banks to underwrite general obligation bonds of municipalities, many of which are sold to finance water utilities. Along with underwriting municipal bonds, banks act as agents to pay the interest and principal on the bonds. Although under the act commercial banks are not permitted to underwrite revenue bonds, they buy many revenue and other bonds for their own investment portfolios and for the accounts of customers; they also fill the role of trustee and paying agent on revenue bond issues.

Several other relationships that exist between banks and the water industry could be cited, but the ones mentioned give an idea of the wide range of these relationships.

Future Needs

Bankers are like any other group of forward-looking business men—they want to know as much as possible about the industries they serve so they can tailor their services to meet the needs of the industries. Frankly, keeping up with the water industry has become a mansize job. In addition to all the technical advancements in the industry, there is such a tremendous rate of expansion that it is difficult to keep track of projected needs.

The government's estimate that the water supply of the United States will have to double by 1980 to meet projected needs is understood to be on the conservative side. Naturally, everyone tries to translate such projections in regard to his own field. In terms of financing, the tremendous expansion in the water industry is going to require more than 1.5 billion dollars each year for the next 20 years to pay for capital improvements. This is a lot of financing for any industry, and the big concern is where the money will come from.

Municipal Bonds

The financing for privately operated water utilities will, of course, be handled through the same channels and by the same methods used for the past 150 years. Those associated with pri-

vately owned water utilities know that the banks, from the outset, have been working very closely with them in filling their needs.

The major source of money for future financing, probably more than 70 per cent, will be raised through the sale of municipal bonds—general obligation or revenue bonds. A banker cannot help but be aware of the rapid expansion in the field of municipal finances. The dollar value of municipal bonds issued in 1960 was almost twice as much as the total issued 10 years ago. This means that when a city, town, authority, district, or commission comes into the market with an issue to finance water utilities, it will be facing stiff competition for the investor's dollar. The investors referred to are not speculators. Historically, the principal buyers of municipal bonds are commercial banks, insurance companies, trusts and estates, and professional investors who are interested in the safety and tax-exempt features of municipal bonds. Astute investors of this type are naturally looking for the best bonds available at any given time. They must overlook those that fail to measure up to the high standards of the current market.

Competition from other bond issues makes it increasingly more important for municipal officials to keep the ultimate investors in mind right from the outset when preparing to market new issues. The investors play a big role in determining the net interest cost a community will have to pay. Bonds that do not measure up to investors' standards can, of course, be sold. The community, however, will pay more in terms of interest rates.

These investors, or lenders, are looking for the strongest security or bond that the municipality, the borrower, can provide. Naturally, the investor is interested in a reasonable return on his money, but at the same time he wants to be assured that he will be repaid. On the other hand, the borrower knows that a strong bond is attractive to security-minded investors and will bring the lowest interest rates. These objectives are not mutually exclusive; they can both be met with a strong bond.

Selection of Bond Issues

With the investor or lender in mind, attention may be turned to some of the factors that should be considered in the preparation and sale of a bond issue. Some of the factors may seem minor, but the more competitive the market the more important they become.

The first steps in any public improvement program, such as arousing public interest to the fact that a need exists and having a consulting engineer determine the most feasible way to meet the need, are well known and need not be discussed here. Once the cost of the needed improvement has been estimated, the next concern is how it should be financed. Unless one is experienced in municipal financing, it will probably save him money in the long run to bring in a reputable financial consultant who specializes in municipal financing. At the same time, one must retain the services of a recognized bond attorney who, through experience, qualifies in this field. The financial consultant will consider all the various possibilities for financing that exist for the particular community, and the attorney will study the legality of proposed financing.

The various types of bond issues essentially fall into two categories: general obligation bonds and revenue bonds. Of course, the features of both

types can be combined in a variety of ways.

General Obligation Bonds

General obligation bonds are strong bonds because the interest and principal are paid from the municipality's general revenues. These bonds are backed by the full faith and credit of the municipality. The funds include income from taxes and all other sources of income available to the community. In other words, the community is pledging its future taxes to pay off the bonds. These bonds combine maximum flexibility for the town in allocating its resources with the most desirable security for bondholders.

Most states and their political subdivisions have a statutory limit on the amount of general obligation bonds they can issue, and, consequently, water improvements will be competing with other capital budget needs. In many communities, this problem has made general obligation bonds unpractical for financing utilities.

Revenue Bonds

Because in many communities general obligation bonds are not practical for financing, more than half the bonds issued for water utilities are revenue bonds. The security in a revenue bond issue is a bond resolution or trust agreement that pledges the revenues or income from the sale of water to the payment of interest and principal on the bonds. Revenue bonds are payable solely from revenues specified in the trust agreement and are usually excluded from debt limitations.

There are other advantages to revenue bonds. In addition to their ability to finance the needs of one community, revenue bonds can be issued for a facility that will provide for two or more political entities, such as two towns, two cities, or two districts. Then, too, as many people probably have discovered, it is usually easier to collect water bills than taxes.

Revenue bonds are not backed by the full taxing power of the community, but they can be attractive to the investor if they have a strong trust agreement or bond resolution as the security.

Trust Agreement

Under a trust agreement, a bank, as trustee, is acting in behalf of the bondholder. It is the bank's job to see to it that the municipality that issues the bonds complies with the terms of the resolution or trust agreement, terms pertaining to the construction of the facility; the carrying of proper insurance; the filing of engineering, financial, and budgetary reports; and the allocation of revenues for the payment of interest, principal, reserves, and other specific purposes. The trustee also sees to it that the municipality fixes and maintains water rates sufficient to meet the necessary expenses and debt service requirements.

One may be advised from time to time that it is not necessary to have a bank act as trustee. In many instances this is true. But with about 30 years of experience in banking and financing, it is the author's opinion, and the opinion of many investors, that if a trustee is not appointed and the municipality itself assumes the implicit fiduciary responsibilities, the bondholders and the community are not getting the same impartial protection that they might get with a bank as trustee. Moreover, municipal officials or members of a commission or authority would do well to consult

counsel as to what responsibilities they might be incurring personally.

Combination Bond

There is a third common type of bond, a hybrid, which combines the best features of revenue and general obligation bonds. The revenues from the water facility are pledged to pay off the interest and principal. If, however, the revenues are not sufficient, the municipality agrees to use general tax funds to meet the deficiency.

This type of bond is often issued by communities that have a good credit rating and want to take advantage of the lower interest rates that usually result from a pledge of taxes, as with straight general obligation bonds. And in some instances, these bonds are excluded from the debt limit.

Bond Sale

Once the type of bond is determined and the issue is ready for sale as far as the legal aspects are concerned, it is a good idea to give some thought to the timing of the sale. The financial consultant will be quite helpful in this respect. He is in touch with municipal bond dealers and will know what other issues are being scheduled. He will also consult with overlapping governmental units to make sure county, special district, or other bonds are not being sold at the same time. If many bonds of similar quality from similar communities, or communities in the same geographic area, are offered at the same time, they will be competing.

Information concerning the sale of the bonds should be distributed well in advance of the sale. In addition to advance publicity in financial and local publications, it is essential to prepare a special prospectus, which should give as much financial data as possible. It should include information on the municipality's debt, its tax rate, its population, its industries, its tax collection records, and any other information that would be useful to underwriters.

With revenue bonds, the prospectus is more important for two reasons: (1) the number of underwriters is reduced because commercial banks cannot underwrite these bonds; and (2) because the investor considers the risk slightly greater, he wants to know more about the project. He wants to know about the engineering studies. He wants to know if the project has any history of producing revenues. He wants to know if anything will affect the income-producing capacity of the project.

The investor wants the names of the consulting engineer, the financial consultant, and the bond attorney. If the project is a new one, the investors rely heavily on these experts. Because investors do not have time to investigate the qualifications of these men, it is essential that they be men of extensive experience in their field.

The official notice of sale should be published at least 10 days to 2 weeks prior to the sale. It should appear in the *Daily Bond Buyer,* a prime source of information about the municipal bond market, and other financial publications. The notice should give the name of the issuer, the purpose of the bonds, and the time, date, and place where the bids will be opened. It should state the name of the attorney who will give an unqualified approving opinion. It should also state the basis for determining the winning bids and give the name of the person to contact for additional information.

The prospectus and notice should be distributed to underwriters, including commercial banks and large institutional investors who might be interested, simultaneously with the invitation to bid. Prior to the publication

of advertisements, all available financial data should be sent to the rating agencies. The more interest that can be generated in an issue, the better.

Of course, when the bids are received, the municipality should decide very promptly if it will accept one of the bids. If so, it should make the award in writing to the successful bidder. Underwriters want to sell the bonds without delay. The bids, in fact, are based on current market conditions. If the underwriter does not sell the bonds quickly, another issue, more attractive to investors, might be placed on the market, and the underwriters would have to sell the bonds at a lower price. The bonds should be delivered, with an approving legal opinion, as promptly as possible.

Interest Payment and Administration

Once the bonds are sold, the main job of the municipality is to make payment of interest and principal on time. For the convenience of bondholders, municipalities usually name a local bank and a bank in a large financial center to act as *co*paying agents. This cooperative arrangement with banks makes it much easier for bondholders, scattered throughout the country, to collect interest payments.

Another step that should not be overlooked is the supply of information. Municipalities should send financial reports on a regular basis to the underwriter who bought the issue, other underwriters, commercial banks, large investors, and, of course, the rating agencies. Underwriters who deal in municipal bonds keep extensive files on communities that issue bonds. A community should establish records of punctual payment and should let the underwriters know about them. This will help maintain, or even improve, the credit rating of the community.

Interest on the part of banks can and should be of help to a community planning a bond issue. With experienced staffs and extensive files, banks can provide important facts that might be helpful. For instance, with revenue bonds, banks can be of assistance in helping to set up a revenue flow when the trust agreement is prepared. They can help answer such questions as: Who will receive the revenues? Who will handle the funds for maintenance and upkeep expenses? Banks can help reduce interest costs during the period of construction by investing proceeds of the bond sale until they are needed for construction. Banks can also invest other monies not needed currently.

Summary

Following is a check list that may be used when a bond issue is to be prepared and sold:

1. Determine the need for the bond issue.
2. Gain public support.
3. Check with bankers.
4. Meet with consultant.
5. Meet with the bond attorney.
6. Determine the best type of financing.
7. Select the trustee and paying agent.
8. Prepare the prospectus.
9. Inform underwriters and rating agencies.
10. Publish an official notice in the *Daily Bond Buyer* and other publications.
11. Award bonds promptly.
12. Deliver bonds promptly.
13. Make payments promptly.
14. Keep underwriters and rating agencies informed.

Public Relations Campaign in a Referendum

Leo Louis

A paper presented on Jun. 7, 1961, at the Annual Conference, Detroit, Mich., by Leo Louis, Pres. & Gen. Mgr., Gary-Hobart Water Corp., Gary, Ind.

THE Gary-Hobart Water Corp. is a privately owned water utility incorporated under the laws of Indiana, regulated by the State Public Service Commission, and serving Gary, Hobart, and Griffith, Ind., together with adjoining suburban areas. The history of this utility parallels the history and growth of Gary, which is known as the fastest growing city founded in the United States in the twentieth century.

Background of the Company

Gary rose with the establishment of a steel plant on the southern tip of Lake Michigan in 1906. The United States Steel Corp. not only built one of the world's largest steel plants there, but also provided for the planned growth of a city. To take care of this growth, it established subsidiary companies, including the Gary Heat, Light and Water Co. This utility company built an intake tunnel approximately 3 mi long from a crib at the bottom of Lake Michigan to a suction well and pumping station approximately 1.5 mi inland. A complete distribution system was designed and installed as the need progressed during the years that followed. The only treatment the water received was chlorination, however, and the water was often cloudy, sandy, and not very palatable.

The water utility passed through several corporate changes during the years, and from time to time the city fathers discussed the possibility of municipal ownership. The big obstacle to municipal ownership was the need for a filtration plant and the tremendous outlay of funds required to build such a plant. Water rates were always low in Gary, as the water utility was maintained as a part of a combined utility holding of electric, gas, and water properties, and the policy of the utility owners was to absorb part of the cost and overhead of the water utility with earnings from the other utilities. In fact, the water rates established in 1912 remained the same for the next 40 years. Gary politicians did not deem it expedient to purchase the property with its low water rates only to be pressured by the citizens to build a filtration plant that would necessitate a substantial rate increase.

In 1951, the water utility was sold to a new owner, the C. S. Mott family of Flint, Mich., and the Gary-Hobart Water Corp. was born. With the complete separation from the combined utility group, the water utility was forced to stand on its own feet. The new owners knew this, and knew also that a filter plant would have to be built. Before the filter plant could be financed, the rates had to be increased and an interim increase was granted by the State Public Service Commission in 1952.

Construction of the filter plant was started in 1952 and completed in 1954.

Operation of the new facility began on May 1, 1954, and the Gary customers were pleased. No longer did they have sand in their bathtubs, soup in their water glasses, or the "chlorine cocktails" they found distasteful.

Steps Toward Municipal Acquisition

Immediately, however, the utility became fair game for those in the city administration who had long considered municipal ownership but had not wanted to take on the expense of building the filter plant. On the day the new plant was dedicated in July 1954, the mayor announced that he intended to initiate steps to acquire the utility. The owners stated that the company was not for sale, and there the matter stood for 5 years, with only occasional rumblings of things to come.

Meanwhile, the utility property continued to grow in the rapidly expanding industrial area. The company had also purchased the water utility in the adjoining town of Hobart, and in 1954, the company joined the two systems with one source of supply. A year or two later, a supply line was extended to the town of Griffith, and water was sold at a wholesale rate to that municipally owned system. Pumpage increased from 4.4 bil gal in 1945 to 8.5 bil gal in 1959. The number of customers doubled in that period from less than 20,000 to almost 40,000. Gary-Hobart Water Corp. installed approximately 140 mi of mains after 1951, including several large reinforcing mains needed in the system. Storage reservoirs of 5-mil gal capacity, together with three booster pumping stations, were constructed. These improvements were part of an overall plan to provide the best of service for the customers and to allow for the future growth of the entire area.

This, then, is the background to a case history of public relations. This paper will not discuss the political, social, or economic aspects of private as opposed to public ownership, but will only describe the steps taken to win a referendum of the citizens of Gary on the issue of whether or not to acquire the privately owned water utility.

Some information on political background is necessary to indicate how people will vote one way in an election and another way when only their own judgment influences their votes. Gary was a Republican community until the 1930's, but has been strongly Democratic ever since, with the exception of one Republican administration. This Democratic trend became more pronounced at each city election and, in November 1959, culminated in the largest majority vote ever recorded for a mayoralty candidate. Gary is a strong labor union city, with the United Steel Workers being the dominant union. This union has been a strong factor in political campaigns. It is also a city with a large Negro population, and this group has also been a strong factor in the political life of the community.

In the late spring of 1959, the mayor announced that he had employed a firm of fiscal experts and a consulting engineering firm to report to the city on the feasibility of purchasing the water utility. Up to that time, the water community had been carrying on a minimum public relations program, at least as far as money was concerned. The company had run occasional institutional ads in the newspaper and sponsored infrequent radio programs, mostly of local athletic events, but had concentrated on good water service to its customers.

The public relations program was probably no better or worse than that of any other water utility. The company had one employee, Ruby Proctor, a former newspaper feature story writer, on its staff to assist in this program. She not only took care of newspaper work, but gave talks to civic and school groups, conducted plant tours, handled customer inquiries, and followed up complaints about high bills.

Surveys and Petitions

The threat of a referendum, however, decided the company to employ a public relations firm. The firm, Bozell and Jacobs, immediately advised the utility to institute a public opinion survey to see where the utility stood with its customers. The first opinion poll was completed in June 1959, and the results were decidedly pessimistic. Sixty per cent of the people were critical of the water rates, and one person out of five was in favor of city ownership, although 39 per cent had no opinion on the ownership issue. One encouraging factor was the indication by 91 per cent of those interviewed that they had a favorable attitude toward the utility's service record.

Still, not much was heard about the issue until after the municipal election in November. The company increased its institutional advertising with the aid of the public relations firm, concentrating on the good-service angle. Immediately after the municipal election, which, as stated above, the mayor won with the largest plurality of votes ever recorded, the mayor announced that his consultants had found it feasible to acquire the water property. He asked the city council to pass the necessary legislation to establish a referendum on the issue. Indiana law permits the acquisition of such property only after a referendum requested by 5 per cent of the registered voters in the city. Petitions were circulated by the administration with the promise of lower water rates, and the necessary signatures were obtained.

The company had no way of preventing the petitions, but the first step in its publicity campaign was to inform its customers of the false assumptions behind the allegations that rates were out of line and that privately owned water utilities were unique in the state.

Another public opinion poll was taken in late November, and the company found that if anything, its position was a little worse than before. The talk of high water rates had apparently made an impression on customers, as 66 per cent were now critical of the rates, and 45 per cent now preferred municipal ownership where only 18 per cent had previously expressed this preference. The no-preference group fell from 39 per cent to 29 per cent.

Company Positions

The city council set Feb. 16, 1960, as the date for the referendum, giving the company 2½ months to reverse the thoughts of the voters. With the approval of the company's owners, it was decided to go all out to convince customers that rates were not the issue; that although rates were higher than in some neighboring cities, they were not out of line with the service provided; that private ownership could best serve the overall good and growth of the community; and that municipal ownership could neither lower rates nor provide better service.

These contentions, stated in various ways, formed the crux of the issue. The public relations firm prepared a series of newspaper ads, most of them full-page presentations of the company's position. These were all headed the same way: "So That You May Know the Truth." The campaign slogan, "Don't Muddy Gary's Water With Politics," appeared as a part of every ad—on billboards, bus signs, newspaper copy, and radio commercials. The company decided to concentrate on the fiscal agent and bond firm that acted as consultant to the mayor. This firm was to receive a fee of 2 per cent of the total bond issue on a contingency basis if the project went through, and the utility stressed that a $500,000 fee would be paid to this firm if the selling price was $25,000,000. Most of the criticism of the entire project was turned on this out-of-town bond firm rather than on the city administration.

One quirk of Indiana law is that the referendum is held before a valuation can be set on the property. The company stressed the point that if the people voted for the proposal, they were signing a blank check without knowing how much they would have to pay.

The fact that the company had a good service record and a good "corporate image" in the eyes of its customers now began to pay off. With rates virtually the only issue, and no complaints of poor quality or quantity of water, the company went directly to the people with its side of the story. The utility had never had to restrict water to any customer, and pointed with pride to the fact that it had built the filter plant and had given Gary good water. Many company supervisors were active in civic groups and service clubs, and these groups were used as springboards of public opinion. The past history of community service gave the company's story the attention and acceptance it deserved. During the campaign, the author was struck by the observation that, although water utilities have historically consisted of silent people working in silent places, people do remember the services they have or have not received.

A number of debates were held between company supervisors and members of the city administration on the radio and before all types of audiences —service clubs, ministerial groups, Sunday school classes, and neighborhood clubs.

All of the company's employees rallied to the cause. This is probably the chief factor in the company's success. They had the most at stake, and they were sincere in wanting to retain the job advantages and job security they possessed. An organized effort was made by employees to persuade their friends and neighbors to vote the right way. The local union of District 50, United Mine Workers of America, which represented the operational employees, pledged its support in service and funds. The president and the secretary of the union were relieved from their normal utility duties, and worked full time in helping to organize and direct the employees' campaign and in contacting members of other unions to gain a share of the labor vote.

Over the signatures of these union officials, post card appeals were sent to water customers asking their support at the polls. Because many voters are not water customers, employees worked overtime to deliver personally campaign literature to apartment houses in the community. Sev-

eral newspaper ads, billboard signs, and many letters printed in the daily paper added weight to the union's effort on behalf of the company.

Groups and Organizations

Citizens' committees and women's groups were organized to oppose municipal ownership. The company asked 20 or 25 women in each of Gary's six voting districts to contact by telephone each family in their district. The women first determined if a family was for or against the acquisition, and if the family was against it, its name and phone number were placed in a card file. The people named in these files, and others whose names were furnished by company employees, were then contacted on the day of the referendum to get out the vote favorable to the company's side. These workers were all volunteers, although the company did offer to pay them for their time.

A citizens' group with the title "The Citizens Committee Against Political Control of Gary's Water" was organized with a local business man as its chairman and a number of prominent professional and civic leaders on its board of directors. This group established a downtown office, conducted its own campaign for funds, advertised on the radio, in newspapers, and by pamphlets its objection to municipal ownership, and dispensed car stickers and "Vote No" buttons.

Many civic organizations came out in favor of the utility. Some worked actively to defeat the municipal drive (Fig. 1). Perhaps the greatest serv-

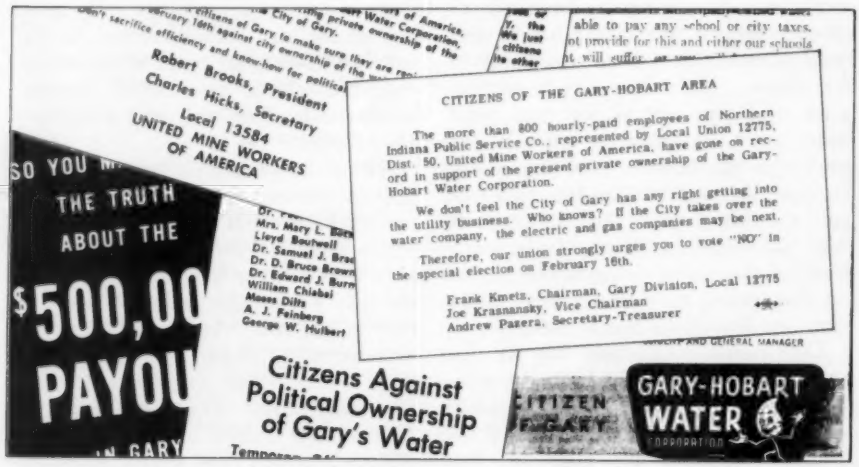
Robert Brooks, President
Charles Hicks, Secretary
Local 13584
UNITED MINE WORKERS
OF AMERICA
THE TRUTH
ABOUT THE
$500,00
PAYOU
CITIZENS OF THE GARY-HOBART AREA
The more than 800 hourly-paid employees of Northern Indiana Public Service Co., represented by Local Union 12775, Dist. 50, United Mine Workers of America, have gone on record in support of the present private ownership of the Gary-Hobart Water Corporation.
We don't feel the City of Gary has any right getting into the utility business. Who knows? If the City takes over the water company, the electric and gas companies may be next.
Therefore, our union strongly urges you to vote "NO" in the special election on February 16th.
Frank Kmets, Chairman, Gary Division, Local 12775
Joe Krasnansky, Vice Chairman
Andrew Pazera, Secretary-Treasurer
Citizens Against
Political Ownership
of Gary's Water
GARY-HOBART
WATER
CORPORATION

Fig. 1. Paid Advertisements and Handbills for Private Ownership

In addition to newspaper advertisements and handbills purchased by the company, civic groups and unions distributed postcards and other material publicizing their position in favor of private ownership.

ice was given by the chamber of commerce, whose governmental affairs and tax research department did an objective study entitled "Private Versus Public Ownership of the Local Water Company."

The junior chamber of commerce also helped the company's campaign. It took a public stand in favor of the utility, ran advertisements supporting its viewpoint, handed out "Vote No" "Vote No" in several languages. The truck with its visual and vocal appeal was also used to lead a parade of utility cars and mobile equipment through the streets of Gary in a final campaign effort.

Newspaper Support

One of the first salvos fired in the campaign was by the editor of a chain of weekly newspapers that covers parts

Your Decision
Here's the Way Water Rates Are Computed
The Company's Side of Water Dispute—No. 1
How Gary Water Change Could Affect Neighbors
(An Editorial)
Our Conclusions On Water Problem
(An Editorial)
Water Problem Conclusions

Fig. 2. Newspaper Reports and Editorials

Local newspapers investigated the referendum issue in detail, publishing many articles on the subject. These studies led them to support private ownership of the water utility in editorials.

buttons by the thousands to steelworkers as they left their jobs, and distributed car stickers with a "Vote No" message at city and industrial parking lots.

An important part of this effort was the use of a sound truck that patrolled the streets in the days before the referendum. Because many nationalities are represented in Gary's population, the truck was painted with

of Gary and its suburbs. He attacked the city's action in an editorial that gave a very sound analysis of the probable cost to the community if the takeover were accomplished. Throughout the campaign this editor struck many strong blows in the utility's favor. Some of his editorials were reproduced and mailed to all water customers.

The daily newspaper and leading radio station did an objective study

of the issue. The Gary *Post-Tribune* sent a reporter to the State Public Service Commission and to a score of cities around the state to investigate the relative value of privately owned and municipal water systems. A daily front-page story dealing with the results of the study claimed public attention for weeks. After this series, the paper came out strongly in support of the utility (Fig. 2).

Radio editorials supporting Gary-Hobart Water Corp. were broadcast several times daily for many weeks. Station WWCA invited both the mayor and the manager of the utility to make use of its facilities on an equal-time basis.

A few days before the referendum, the mayor and the author were invited to take part in an interview program on one of the Chicago television stations. Although this part on the program received little advance publicity, public response was reported to exceed any previous issue presented by the program. From later reports, comments favored the privately owned utility by a ratio of 5:1.

Two weeks before the referendum a third public opinion poll was conducted. The results were startling. Those interviewed now indicated a preference of 60 per cent for continued private ownership, 15 per cent for municipal ownership, and 25 per cent undecided. The company was elated with this change in opinion, and this spurred on its efforts.

On referendum day, only a skeleton crew operated the filter plant, and the offices were closed while employees worked as poll watchers, drove voters to the polling places, did baby sitting, and manned the telephones calling the thousands of potential vote-no citizens to remind them of the issue at stake and to offer service if needed to get them to the polls. Employee poll watchers were certified by the newspapers and radio station.

Results of Referendum

The result of the voting was overwhelmingly for the company. By a vote of 26,539 to 10,723, or a ratio of more than 2.5:1, the issue was decided in favor of the present owners.

The results of the referendum indicated the value of telling the utility story to the people and having the people believe it. Gary citizens believed the facts given them by the utility because the company and its employees had a good reputation in the community and because it had given them fine-quality water and good service at a fair price. Good public relations had been maintained on the organizational level with at least one supervisor an active member of every civic or service club in the city. Utility personnel were encouraged to participate in community projects.

It is true that a considerable sum of money and a lot of time at every level of employment was spent on this campaign. This effort and money were not entirely wasted, however, as the customers of Gary-Hobart Water Corp. probably know more about their water supply system and organization than the customers of any other water utility in the United States. The issue was on everyone's tongue for 3 months and people still talk about it. This was a crash public relations campaign that will show results for years to come. Employees of the company are cognizant, however, that they must continue to give customers the same quality of service to retain their confidence and friendship.

Regulation of Privately Owned Water Utilities

J. J. Barr

A paper presented at the 1960 Annual Meeting of the National Association of Railroad & Utilities Commissioners, Las Vegas, Nev., by J. J. Barr, Pres., American Water Works Service Co., Philadelphia, Pa.

REGULATION applies to as many water companies as it applies to privately owned utilities of other types. On occasion, some people associated with the water supply field get the impression that water companies are poor relations among utilities, overshadowed by the electric, telephone, and gas companies. Water companies are required to comply with essentially the same rules, principles, and requirements of regulation that apply to, and have apparently been promulgated for, operations that are much larger in number of customers, invested or available capital, revenue per customer, or extent of justifiable administrative personnel.

This observation must not be taken as reflecting a defeatist attitude on the part of the water utilities, for they accept these conditions as part of their undertaking to provide the most essential of public services. Nor do water utilities quarrel with the necessity for basically uniform principles of regulation. Instead, water utilities continue to strive and hope for a better appreciation of the effect of constantly broadening, yet tightening, principles on the comparatively smaller and older regulated water companies.

Attitude of Investors

The attitude of investors has a relation to the regulation of utilities. The comments in a recent pamphlet [1] generally reflect the attitude of investors toward the water supply industry.

In the opening paragraphs of the pamphlet, mention is made of the great demands for increased water supplies in the United States at present and in the immediate future. From statistics and information furnished by AWWA, the pamphlet presented, among other items, the following observation:

> Increasing demand for water is by far not the only problem faced by the water utilities. Although the water companies are getting somewhat more consideration from both the general public and the regulatory authorities, rates are still regarded as relatively unrealistic. The American Water Works Association points out that, in many cases, water utilities are undercapitalized and are thus hard-pressed to keep up with impending large capital investments. Today, the average cost for water service per person per day in the United States is 3 cents. To meet present deficiencies and the demands of population growth, AWWA believes this figure should be increased to at least 5.5 cents. Even at that figure, water service would still cost far less than electricity or telephone [service].

The article also stated that "low water rates have, historically, provided an unfavorable comparison of water utilities with electric and gas counterparts."

Accounting Requirements

Any remarks with respect to the regulation of water utilities must at the outset include a mention of the inability of water utilities to gain an adequate rate of return. The effects of depreciation and the need for a greater allowance for the real fair value of plant and equipment are not sufficiently recognized. Regulation poses other problems, however, which have not received the attention they deserve. These problems will be discussed here.

For example, in the regulation of water utilities, size and age of a utility should be considered carefully. A major part of the continuing investment in the properties of water utilities was made many years before uniform systems of accounts were imagined, and even long before the introduction of double-entry bookkeeping. As a consequence, water utilities sometimes cannot, in their accounting records, comply completely with the many refinements of the present-day systems of accounts in support of a rate base and other purposes involving investment in properties installed as long as a century ago. True, an alternative of sorts is available, because utilities can offer estimates of costs. But the management of the small utility in particular is frequently confronted with a dilemma in its search for a justification of, and the cash for, payment of the high cost of engineering estimates. Then too, engineering estimates of the cost of building dams or installing underground pipelines 50–100 years ago will inevitably produce amounts somewhat below recorded cost. Confronted with such a result, it is often difficult to accept the basic idea that the admittedly sketchy records maintained by the old-school treasurer or secretary were so far wrong and suspect. Instead, it may be wondered if today's estimators really know all the problems and costs actually incurred so long ago.

Yet, it is not unusual to find in rate decisions that such estimated costs are substituted for recorded book cost, almost entirely because nineteenth-century managers did not properly or fully anticipate the accounting requirements of the twentieth century. This is an example of one problem of the water utilities that, in the author's opinion, can be directly assigned to their age.

In a recent rate decision, a computed depreciation reserve requirement was substituted in the rate base elements for the actual book reserve. The computed reserve exceeded the book reserve. Evidently the major part of the difference arose from the accumulation of the book reserve over more than 50–60 years and from the application of depreciation concepts entirely different from those applied in the reserve requirement study. Through those early years, management reasonably complied with the principles of their time for providing depreciation, but had not prophesied and applied the twentieth-century approach in the nineteenth century. This illustrates how the application of modern principles of regulation seriously affects water utility properties with their proportionately greater attained age and long service life.

Mergers

Mergers may cause another adversity of attained age in water utility regulation. A great many, if not the majority of water utilities with which the author is acquainted, are still small, but grew through corporate mergers or reorganizations of yet smaller entities. In almost every such merger, it

is easy to see that the smaller units had reached the point at which they could not, by themselves, attract capital and maintain credit sufficient to meet demands for rapid expansion as their service was recognized and accepted as a necessity and as the population of the communities they served increased. A great number of these mergers took place before the establishment of regulation, and many were accomplished relatively soon after the start of state regulation. Many mergers were effected only after having obtained regulatory approval.

In these mergers it was not uncommon—instead it was, apparently, accepted practice—to issue securities for the new or continuing company in amounts higher than the principal or stated value of the outstanding securities of the predecessor companies. Evidently this was done in recognition of the values of the properties involved at the date of the merger. Application of the accounting rules and regulatory principles pronounced at a much later date has resulted in such excess amounts generally being classified as "utility plant acquisition adjustments." The amounts so classified and the amounts representing amortization thereof are almost uniformly and without exception disallowed and disregarded in water utility rate proceedings.

If, in such cases, regulatory commissions gave their approval to the capitalization of the new company and to the issuance of their securities, it is reasonable to assume that the regulators found virtue and merit in granting to investors a recognition of their present-day values. Had the men who managed these utilities at the time realized that the inherent obligation they undertook to investors would, under today's principles, result in surcharges for the investor, it is quite possible that the mergers would never have been effected. As a result, service to the public would have suffered, and the growth and welfare of the communities involved would have been hampered.

There are many small water systems in existence today which, if merged into or consolidated with a larger operation, could improve service and most probably lower rates. But such mergers and consequent improvements are prohibited by the inability of the larger entity to offer reasonable recognition of present-day values in the acquisition of the problem child. This prohibition, of course, arises from the accounting requirements and rate base treatment mentioned above.

Although space is insufficient to provide examples of this problem, the author foresees increasingly knotty and frequent involvement for both management and regulation as more and more of the real estate developers' so-called water systems demonstrate their inherent inability to provide reliable residential service, let alone fire protection.

Regulation and Small Utilities

Comparatively speaking, water utility operations are small, as far as regulation is concerned. Most people will agree that preparing financial statements and reports for entities whose balance sheet items are expressed in millions does not involve much more work than preparing statements with items in hundred thousands. Likewise there is not much work difference between preparing all the necessary documents for the issuance of $500,000 of mortgage bonds and preparing the documents for $5,000,000 of bonds—including that part of the program that involves regulatory approval for their issuance. As the workload is not

much different, it normally follows that the cost will not vary greatly between the large and small transaction. But these are relatively simple examples of the effect of size on cost. Two other examples will be considered.

The amount of money that a small water utility must spend to prepare and present an application properly for a rate adjustment, in conformity with all the established principles of rate making, is often shocking. This is particularly true when viewed in relation to the utility's revenue per customer, its total revenue, or the new income to be derived from the rate increase sought. Certainly corporate management is uncomfortable in the realization of this fact, but apparently there is no choice in the matter.

Many current rate decisions rather clearly indicate that a water utility has been denied a substantial part or even all of a rate increase primarily because the utility did not go to sufficient extremes in substantiating the merits of its position. Even if an increase is granted, all too frequently the rates finally established do not accommodate for the actual cost incurred by the utility in presenting its position. Consequently, those costs are not fully recovered, as they should be.

The average small water utility that appears before a regulatory body today in pursuit of a rate increase without the services and extended studies of an independent consulting engineer, an independent rate-of-return expert, and representation by attorneys who are fully skilled and experienced in ratemaking matters, runs a substantial risk of being told—possibly 6 months or a year later—that because of the utility's failure to assume properly and fully the burden of proof, the utility is to be deprived of a part or even all of the additional earnings to which it considers itself entitled. The reason for disallowance will not always be given in so many words. Instead the company is likely to find such expressions as "lack of probative value," "unsupported opinion evidence," "best evidence available in the record."

Experienced and qualified independent water works engineers and rate-of-return experts whose opinions will be respected enough to offset the inherent risks mentioned above, and attorneys experienced in rate matters, are far from plentiful. They rightfully demand high rates of compensation. The result is that if the utility undertakes to protect the interests of its investors properly—not to mention the interests of its customers—the utility must anticipate expenses of $25,000 or $30,000 in the presentation of an application seeking an increase of only $75,000 in its revenues. Stated in a different way, the cost of the presentation of its rate increase application may amount to more than the annual operating income the small water utility hopes and seeks to obtain.

Consequences of High Application Cost

The same water utility, assuming it is allowed to amortize its rate case expense over a 5-year period, will probably be forced, because of added investment in its property and increased cost of operation, to apply to the regulatory commission for further rate relief before it has accomplished the amortization of the cost of the last case. On this point quite a number of rate decisions say, in effect, that the utility must forget that part of the cost of its last case which was not amortized; instead, the utility is only entitled to recognition of the cost of the current proceeding. If this penalty, as it may be termed, represented

only a minor part of the income to which its investors were entitled, as might be the case with a large operation, then very little attention or sympathy is deserved. But such is not the case with many small water utility operations.

The high cost of rate proceedings to most water utilities produces still another highly undesirable result from the viewpoint of the utility and, the author believes, from the viewpoint of the regulatory body. This high cost is one of the contributing factors to the high percentage increase in rates that small water utilities must seek in one proceeding. An electric company or telephone company before a state regulatory body rarely seeks as much as a 20 per cent or 25 per cent increase in its rates. On the other hand, it is almost normal for an application for increase in water rates to involve at least a 25 per cent increase. Increases of 50 per cent are not infrequent. Many small water companies would go broke paying rate case expenses if they attempted to obtain approval of justified rate increases in 10 per cent increments.

Conclusion

The following impression may be inaccurate, but the author is inclined to believe that water utility rate cases attract far greater and less intelligent opposition than do the major cases involving the larger utility operations. The author knows of a medium-size water utility that, as the result of a rate application, received from the regulatory body an order of decision stating that the proceedings had produced the largest transcript and record that the commission had ever made in a single case. Another water utility, whose application for a rate increase was filed in 1957, has still not received a final decision, notwithstanding extended hearings and rehearings before the state regulatory body, appeals to lower courts, and appeals to the state supreme court. Right now it looks as though the same case is headed back to the supreme court. It is, incidentally, a case in which the regulatory commission granted an increase very closely approximating the full amount sought.

It is not the author's intent to imply criticism of regulation, regulators, and regulatory processes. Water utility management realizes that a great part of the regulatory principles is established initially by legislative action and statutes, and that those statutes are frequently interpreted, modified, sometimes even massacred by the courts. Management also realizes that regulatory bodies are frequently hampered by budgetary appropriations insufficient to attract and retain sufficiently large and qualified staffs. Nevertheless, a high degree of dedication to public service prevails among the men and women of those staffs.

Consideration and recognition was given to water utilities in the recent revision of the NARUC Uniform System of Accounts. Water utility managers also realize that some of their problems are of their own making.

Water utilities intend to continue the improvement and expansion of their service, and to accept the problems in regulation as part of the challenge. Regulation is as important to the healthy survival of water utilities as it is to the protection of the welfare of the public.

Reference

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Texas Water Need Forecasts

Homer A. Hunter

A paper presented on Mar. 16, 1961, at the Texas Water & Sewage Works Association Short School, College Station, Tex., by Homer A. Hunter, Cons. Engr., Dallas, Tex.

ACCORDING to *Engineering-News-Record,* in the last 10 years water consumption increased from 79 to 116 gpcd in one city, in another from 72 to 159 gpcd, in another the increase has been from 144 to 231 gpcd, and in yet another from 158 to 185 gpcd.[1] These figures sound familiar although they are quoted from an article dated Apr. 18, 1901. They report water consumption increases for the cities of New York, St. Louis, Pittsburgh, and Washington, D.C., during the period 1890–1900.

This was, of course, before most water supplies were metered, and water waste was very high. Many cities, however, face the same dilemma today that they faced 60 or 70 years ago—increasing demands for water from systems that were designed for considerably less water delivery capacity.

Water Need Surveys

Surveys to determine future water needs may include many facets of the subject, including (1) water supply source, (2) transmission main from the water supply source to the city, (3) treatment plant, (4) high-service pumps, (5) distribution system, (6) elevated and ground storage reservoirs. A detailed discussion of future water needs for any one of these phases would require as much space as is available for this article, because each one requires a different approach to obtain a solution. It is, therefore, necessary in this article to cover as many points as possible in generalized terms.

Factors in Consumption

For Texas cities, many factors affect the consumption of water and the forecasts of future water needs. Among these factors are:

1. The location of the community; whether it is in the semi-tropical southeast corner of the state where annual rainfall is rather high or whether it is in the semiarid far western part of the state. Obviously, people living in cities in the high rainfall belt will require less water for domestic use (including irrigation) than will people in the portion of the state where rainfall is low.

2. The amount of industrial consumption in the city. For example, at Marshall, one industry uses nearly 40 per cent of the city's total water production. Some cities may have similar characteristics. Other cities have substantially no industrial consumption, and all the water produced is used for domestic purposes.

3. The age of the city. Some Texas cities that have experienced rapid growth in the last few years have many hundreds of new homes where the people are planting lawns and using a great deal of water for irrigation. In these cities, domestic consumption is

rather high. On the other hand, older, more stable communities with few new homes use considerably less water.

4. The character of the residential area. Large residential tracts will require more water per capita daily than will apartment areas in the cities.

5. The comparative wealth of the community. Cities of a high economic standard, where people generally have plenty of money to spend, will use more water than towns where the economic level of the community is lower.

6. The water rate structure. In general, the higher the water rate the less water the customers will use.

7. Metering. If the water system is fully metered, the per capita daily consumption will be considerably lower than if all the customers' services are unmetered. This is a point that requires fairly little attention these days, because almost all cities have all customer services metered.

8. Water quality available from the water system. In general, the better the quality of water, the higher the consumption.

9. Type of sewer system. If a community has a sanitary sewer system, water consumption will be 20–25 per cent higher than if no such facility is available.

Domestic Consumption History

With regard to average daily consumption of water for domestic purposes, it might be well to review what has happened in the last several years. In 1905, William P. Mason [2] stated that the following amounts of water are required for domestic consumption: for drinking and cooking, 1 gpcd; laundry, 6 gpcd; bathing, 7 gpcd; water closets, 6 gpcd; flushings, wastes and other miscellaneous uses, 5 gpcd; total, 25 gpcd. Mason added that any use beyond this amount is, in his opinion, very wasteful.

Compare these figures with present-day amounts: drinking and cooking, 5 gpcd; laundry, 20 gpcd; bathing, 10 gpcd; toilet, 15 gpcd; other miscellaneous uses, 10 gpcd; distribution system leakage, 10 gpcd; total, 70 gpcd.

Water consumption by individuals has increased materially in the last few years with the advent of many new water-using appliances and devices. How many people use 3–4 gal of water to brush their teeth twice daily when one glassfull would be ample? How many have run 45 gal through an automatic washing machine to wash a couple of T-shirts? The ordinary toilet in the bathroom uses about 5 gal for each flushing, but manufacturers continue to make the water tank the same size year after year, although 2–2.5 gal is an ample amount to flush domestic wastes down the sewer. Residential air conditioning, the automatic dishwasher in the kitchen, and the garbage disposer in the sink are water users and water wasters. Although all these things contribute to a higher standard of living, waste could be avoided and these luxuries retained.

Operation Data

In 1957, the JOURNAL published "A Survey of Operating Data for Water Works in 1955." [3] This article included a formidable amount of data regarding municipal and private water system operations. A statistical analysis of these data was published in the JOURNAL later that year.[4] It showed that for cities of 10,000–500,000 or more people, the mean water consumption varied from 123 to 147 gpcd. The

average in 1955 was 137 gpcd, as compared to an average in 1950 of 138 gpcd and in 1945 of 125 gpcd.

This statistical analysis also details the data by regions of the United States; the mean consumption varied from 109 gpcd in the South to 220 gpcd in the mountain states. The variation in consumption between small and large cities is not large, but consumption varies a great deal between the various sections of the United States. In this analysis, Texas was included in the South, where the minimum required quantity of water shown in the data is 45 gpcd and the maximum is 275 gpcd, with an average of 109 gpcd.

Variation Between Texas Cities

Each city deserves particular study, and comparisons of cities are sometimes misleading unless a great deal of judgment is used. In Sweetwater, consumption was 121 gpcd in 1952, was 157 gpcd in 1955, and had increased to nearly 195 gpcd by 1960. The city has some industrial use, and is in western Texas, a rather dry area of the state. Statistics show that in 1955, consumption in Grand Prairie was 84 gpcd, and at Irving, consumption was 103 gpcd. These two cities are adjacent to each other and neither of them has any substantial amount of industry. At that time, however, Irving had more new dwellings than did Grand Prairie. This probably accounts for the higher water consumption in Irving, because both purchased water from Dallas, and the quality was identical.

Water consumption at Wichita Falls has increased from 86 gpcd in 1940 to 92 gpcd in 1945, 93 gpcd in 1950, 106 gpcd in 1955, and 123 gpcd in 1959. The average annual increase in consumption has been approximately 2 gpcd. In the far western Texas town of Littlefield, where water rates are low and the people in a fairly high economic bracket, consumption was 152 gpcd in 1953 and 151 gpcd in 1954.

Water consumption records for Dallas are fairly complete, and have been accurately kept for many years. An analysis shows that over a good many years, the domestic consumption amounts to approximately 65 per cent of total water consumption in the city. In 1920, the daily water consumption was 36 gpcd for domestic use. This figure had increased to 55 gpcd by 1930, 63 gpcd by 1940, 78 gpcd by 1950, and was approximately 98 gpcd in 1960. The conclusion may be drawn from these figures that domestic consumption in Dallas has increased almost threefold in the past 40 years, or at an average annual rate of 1.5 gpcd. Estimates indicate that the Dallas domestic consumption figure will likely increase from the present 98 gpcd to 130 gpcd by the year 2000. Total water consumption in Dallas in 1920 was at the rate of 55 gpcd, and has increased to approximately 150 gpcd in 1960. It is expected that total consumption in Dallas will increase to about 200 gpcd by the year 2000.

Data for Forecasts

In planning a water supply or any phase of the water supply or distribution system, it is necessary to research all available data, as forecasts of future population and future water consumption should be as accurate as possible. This research and forecasting is most important in planning water works improvements. Obviously, forecasts that are too low will cause the facility built today to be outmoded before it is paid for. Forecasts that are too high

will cause the investment of funds for facilities whose capacities are not actually needed for many years.

When population forecasts and estimates of future average daily water consumption have been completed, a good start has been made in determining the needs for improvements in a water system. Unfortunately, the customers do not use water at the average daily rate on any single day in the year. They want all the water that they can use when they open the spigot at any hour of the day or night. The water utility must have it there for them.

Forecast by Neighborhood

Forecasts of average and maximum daily water demands for the entire community are necessary to determine the type and size of the water supply facilities. The design of distribution facilities requires detailed studies of each individual neighborhood in the city to determine such factors as present and probable future land use—industrial, commercial, or residential—population density, probable changes in the development pattern, type of residential dwelling in the area—single family, duplex, or apartment—and types of industries likely to come.

These detailed studies concerning neighborhood development are necessary to make estimates of present and future water demands so that storage, distribution, and pumping facilities may be provided in advance of actual needs. Such studies are customarily a part of a comprehensive city plan or master plan, and they are of inestimable value in programing and planning future additions to the water distribution facilities.

Estimating or forecasting rate of growth in each neighborhood in the city must be done so that a time schedule for constructing new facilities can be established. When the time schedule is determined, financing of the improvement program can be arranged. If planning is done in advance, then the water facilities construction can be done in an orderly fashion, and often the financing can be done from current earnings. If the improvement program has to be done to catch up with development, however, then it may be necessary to finance the program by the more expensive method of issuing bonds. For the overall long-range good of the community, sound advance planning for water facility improvements is indispensable.

Consumption Rate Variation

If the city uses a surface water supply, the reservoir and drainage area must be large enough to supply the average daily demand throughout the driest series of years that may occur. The pumps and pipelines delivering water from the source to the town must be large enough to deliver the needed amount of water on the maximum day of consumption. This rate may be 200–500 per cent of the average daily consumption, with a usual range of 200–250 percent. The treatment plant must be able to treat water at the rate required on the maximum day of demand.

High-service pumps at the pump station, distribution mains, and elevated tanks must have sufficient capacity to deliver water to the customers at the maximum hourly demand rate. This rate may be 400–1,000 per cent of the average daily rate, with a usual range of 400–500 per cent. For example, if the average daily consumption is 1 mil gal, then the high-service pumps and distribution system must

be able to deliver water to the customers at the rate of 5–10 mgd. This high rate may be required for only a few hours in the afternoon and early evening, but the facilities must be available for delivery of the water at these high rates when the customer demands it.

If the utility uses a ground water supply, then wells must be able to produce water at the rate required on the maximum day of consumption. The distribution system and elevated tanks must supply peak hourly rates the same as with a surface water supply.

Forecast Challenge

Every city, large and small, has the problem of meeting the ever-increasing demand for water for domestic as well as industrial use. Communities that are aware of this responsibility will survive, and others may deteriorate and disappear.

Forecasting the needs of a community is a rather involved study requiring considerable experience and maturity of judgment. Each city has its own particular pattern of water consumption. In 1961, the usual average daily demand is 50–200 gpcd. Water consumption has increased in the last 40 years at an annual rate of 1–4 gpcd, and this rate of increase may be expected to continue into future years. The maximum has not yet been reached. Maximum daily consumption approximates 2–6 times the average daily rates; maximum hourly consumption rates are 5–10 times average daily rates.

Long-range planning for water works improvements is an absolute necessity if the system is to be operated and expanded in an orderly and economical manner. The customer usually knows only one thing about the water utility: water runs out when the spigot is opened. He has no idea of what is behind the spigot. Planning for improvements to the facility is, therefore, the full responsibility of the water superintendent. This responsibility is a challenge and an opportunity.

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Survey of Operating Experiences Under New State Water Laws

Task Group Report

A report of Task Group 2460 R—Operating Experiences Under New Water Laws, presented on Jun. 8, 1961, at the Annual Conference, Detroit, Mich., by George E. Ferguson (Chairman), Division Hydrologist, US Geological Survey, Arlington, Va. Other members of the task group are Harvey O. Banks, John C. Detweiler, and Robert M. Dixon.

THE scope of this report was developed from a literal and somewhat arbitrary interpretation of the task group subject. The term "new water laws" was arbitrarily taken to mean those enacted during or after 1955. This helped to assure consistency in analyzing data from each state. The term "operating experiences" was defined by the task group as those gained by public officials charged with the administration of state water laws. More specifically, what degree of effectiveness did this experience show the laws to have in coping with water problems and assuring best development and use of water resources?

The task group members made two series of contacts with state officials. Early in 1960, letters seeking general data on the subject resulted in replies that gave valuable background knowledge. A questionnaire requesting more specific and current information was issued in February and March 1961. The response was gratifying and highly informative. Fortunately, a third major source of detailed information was available: this was the recently published statements by state governors to the Senate Select Committee on Water Resources.[1]

New Water Legislation

During or since 1955, at least 26 states have enacted water laws having a significant impact upon the pattern or effectiveness of operation of state agencies. The degree and scope of influence of such laws varied greatly. Existing agencies in at least seventeen states were given greater authority or effectiveness through such laws. New water agencies were created in fourteen or more states. It is significant that many of these new agencies are charged with both surface water and ground water administration as integral approaches to single modern water problems. Seven states reported some consolidation of existing agencies. In many cases, two or more of these reported changes were in the same state.

Legislation in at least eight states gave better identification of both surface and ground water rights. Five others reported strengthening the definitions of surface water rights only, and two additional states indicated that ground water rights only were clarified. No particular geographic patterns of legislation were evident from these reports. Activity in the East matched that in the West.

More effective control or management of overall water resources was evident in at least eight states since 1955. Seven others gained such increased controls over surface waters alone and two more over ground waters alone.

Fifteen states are known to have improved the legal structure at state or local levels for the development of water resources. Six other states did so for surface waters alone. It is likely that this type of legislation was stimulated by opportunities for development made possible by new legislation at the national level.

In at least twelve states, greater authority for the conservation or protection (as from pollution) of both ground and surface waters was legislated. Six or more additional states gained such authority for surface waters alone.

Stimuli to Legislation

Droughts are known to be catalysts in the development of new water legislation. State machinery for administration of waters is best tested when water supplies are at minimum levels. But this mechanism can be no better or more adequate than its legal base. Thus drought experience serves not only to guide the design of new water legislation but also as a motivating force in enactment.

During 1954, the southern part of the United States was under general drought conditions. In 1955 and 1956, the drought persisted in several large areas and spread to Florida. It was generally broken in most of the southern areas in 1957, but deficient runoff reached drought proportions in New England by the end of that year. In 1958, there were no large drought areas, and runoff was excessive in many places. Since that time, drought conditions have returned and persisted in parts of the Southwest.

Water legislation in many states can be traced to these drought conditions, although a number of states within the dry areas showed little activity. Some of these, particularly in the West, had already attained maturity or stability in water legislation. In others, largely in the humid East, water shortages were not so widely critical as to stimulate new legislation.

Operating Experience

The task group has found that accounts of actual operating experience with state water laws are rarely published. Moreover, many state officials tend to be understandably discreet in making critical evaluation. One western water official indicated a reluctance to comment critically because of the "grave political significance" of local water laws. Latitude for evaluation of operating experience appeared to be greatest in the states where laws were newer and where waters were not extensively allocated. Largely for the above reason, the task group has chosen not to identify the public officials who made specific comments on their operating experience.

A review of the reports of operating experience revealed a number of common water law features that strengthened or weakened operational effectiveness. These features are identified by the headings in the analysis that follows.

The task group also found that a study of amendments to water laws revealed, by the nature of the amendment, the weaknesses of the basic law. Findings obtained in the limited time so spent on the recent statutes are incorporated in this report.

Study Commissions

The use of study groups to recommend new or major revisions to water laws is well established. Such groups appear to be broadly useful. An eastern state official comments, "A legislative interim study committee can discourage organized opposition to proposed water legislation by holding field hearings throughout the state over a 1–2-year period."

Between 1955 and 1957, such study commissions were active and produced reports in at least 24 states.[2] The typical commission was created by a legislative act or resolution, made specific recommendations for water laws, and reported directly to the legislature. Many of the reports included hydrologic and other background facts. One commission has not been productive. The legislature that created it in 1959 has thus far not appropriated the funds by which it could become active.

Administrative Flexibility

There are signs that patience is necessary on the part of water officials, because water laws often are steeped in detail and lack proper objectivity. This has resulted in less than flexible administration. It has required frequent amending of statutes and delays in enacting such amendments.

A study of one amendment reveals that the control agency must have felt unduly restricted in "permittting or prohibiting," without qualification, certain water control structures. The amendment gave relief by the addition of the words: ". . . upon such terms and conditions as it may prescribe." Another amendment was broadened specifically to permit the administrative agency to make an investigation on its own, whereas formerly it ". . . on hearing, shall consider all the pertinent facts . . . and shall determine . . ."

Water laws sometimes set the costs of fees or permits. A series of amendments thus appears inevitable in a period of inflation. Specific dates may also freeze situations. One law prescribed the procedure in dealing with ". . . lands which on Jun. 23, 1948, were declared critical . . ." When amended to ". . . which on the date the area was declared critical, . . ." the administration of the statute was simplified and permitted the qualifying and disqualifying of lands on the basis of current conditions.

In the rapidly changing water situation, even the more objective and general water laws are not always adequate to administer new needs and problems arising every year or so. Flood plain zoning requirements and the need for early acquisition of future water storage sites in areas under heavy growth are examples of special problems that arise quickly.

Newly enacted federal laws also often require companion or enabling legislation at state levels. The federal soil bank plan, for example, was reported as requiring an amendment to a state water code.

Unity of Water

Operating experience appears to show overwhelmingly the desirability of legislation by which the waters of a state may be administered as a unit. One evidence is seen in the sizable number of water agencies called "Division of Water Resources," or some similar name, that have been created under recent water law. It is not surprising that many of these are in the East and Midwest where waters are not already under extensive allocation by long established agencies.

Yet there is little evidence that many older state agencies have lost long and well established water functions to the new bodies. More often the "Division of Water Resources" has been given specific and often broad responsibilities new to the state, and a separate board has been established to coordinate the water activities of all agencies.

In many states, a sizable number of agencies have direct interest in one or more facets of water resources. Comments by state officials and the pattern of new statements attest to the value of coordinated effort. In one state, a new ground water code was written without integration with the existing surface water code, and conflicts appear likely. In another, an amendment is being proposed to the legislature to expand the membership of the board to include representation from additional agencies having water resources interests. The objective is better coordination and administration. Other states report, "The diversity of responsibility . . . does create a need for effective coordination, . . ." and "Although . . . advisory groups have been organized to assist in coordination and cooperative effort, . . . there still remains a need for more effective overall coordination."

Water Districts

A number of state officials express the need for participation in local planning, development, and management of water. Local boards are often organized around basins or other areas that have unity of objective. One official states: "The inclusion of representation on a local level as a cooperating partner in planning and development programs should strengthen the effectiveness with which state agencies will be able to cope with present and future water resources problems."

Another official in a state which, he notes, "has had for many years what we feel is a very good water law" adds, as he comments on further needs, "A watershed district law should help in developing small programs in certain areas."

The experience of a third state, beset with the water problems associated with rapid population growth, is interesting. The state law makes provision for regulatory water districts but does not authorize planning, constructing, or financing through taxation. The state official observes that "without funds to operate, these districts have little appeal when the problems at hand call for drainage and control works."

Provision for Fact Finding

As might be expected, water law administrators are sensitive to the need for adequate hydrologic, economic, and other scientific knowledge. Although the legal framework appears generally to support this need, weaknesses in the laws of some states were cited. These centered around legal requirements for the filing of well logs by drillers. The laws of at least two states were recently amended to require such logs. Legislation is pending in at least one other state. A need for well log filing is expressed by officials in two other states.

Knowledge of the pattern of water use is becoming recognized as basic to the enactment or the major revision of water laws. In one state, further consideration of a major revision of its water laws has been deferred until a statewide study of water use has been made.

Hydrologic facts have long been recognized as one of the foundations for effective water law. One official reported that a committee formed by

the state chamber of commerce recommended "that certain hydrological studies be completed before any drastic legislative change be made."

Priority by Type of Use

The matter of special rights to water by type of use has long been controversial—and, quite naturally, of considerable interest to the public water supply industry. Some state water laws exempt certain types of water use from legal control. This may be the result of legislative compromise. On the other hand, it may be in deference to the impracticability of administering certain types of use.

Little comment was received regarding use priorities under operating experience. An official of an eastern state believes that agricultural use, now exempted under the law, should be included. In 1957, the legislature of a western state amended its ground water law to give preference for domestic use over other uses, and established agricultural use as having preference over industrial uses. Bills being introduced in the legislature of an eastern state would exempt individual domestic use from legal control but would give public water supply the highest priority in controlled use in protected water-deficient areas.

Miscellaneous

Water rights are administered with some degree of apprehension or restraint in at least three states because of the chance that the statutes will be declared unconstitutional. Comments from these states are: "The 1955 revision of . . . water statutes . . . has not been tested in the courts," and "The ground water law passed . . . in 1957 does appear to have a constitutional conflict." Another comment was, "The last ground-water enactment was made in 1953. No attempt has been made to enforce the ground water code—statutes have not had test in the courts."

It is apparent that uniformity in water law is limited. The following comments by state officials speak for themselves: "A legal act must be founded on specific need . . . what is good for one state is not necessarily good for another." "The laws of the West are not suitable for this midwestern state." "Water statutes must be designed around local water use, customs, and history."

Conclusion

Operating experience will undoubtedly continue to be a major force in the rebuilding and amending of state water statutes. Water laws which are found adequate today seem destined to be rendered inadequate by future events. New items such as supplemental irrigation or flood plain zoning emerge and challenge older patterns of water and land use and existing legal foundations thereof. Population growth and a rising economy bring greater competition among types of water use and ever more complex and demanding water problems.

The state water officials are most likely to feel the first and heaviest impact of these new events and forces, and the experience gained will shape the water laws of tomorrow.

Recommendation

It is evident that the responsibilities and activities of the states in the field of water resource development are rapidly expanding, not only in the administration of water rights, but also in

the planning, financing, construction, and operation of projects. There is also an increasing degree of cooperation between state and federal agencies, in planning as well as in financing and repayment for federal water projects. During 1961–1962, it is likely that state officials will collectively gain a considerable amount of operating experience beyond that already reported to the task group.

It is recommended, therefore, that the task group continue for another year. During this period it would make further contacts with state and other public officials in search of more and newer experience data. This first report of the task group will be brought to the attention of these officials. It should give guidance as to the type of experience desired, and may be of sufficient local interest to encourage contributions to the subsequent report a year hence.

Acknowledgments

The task group members are grateful to the state water officials who have kindly responded to the early letters of inquiry and to the recent questionnaire. These officials have contributed experience data that have made this report possible.

The task group wishes to express its gratitude to Harry E. Jordan, the initial chairman of the task group, who made the first series of field contacts with state officials and laid plans for the analysis of material and the preparation of the report.

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Supervisory Control Systems With Leased Telephone Circuits

R. L. West

A paper presented on Oct. 21, 1960, at the New Jersey Section Meeting, Atlantic City, N.J., by R. L. West, Application Engr.—Control Systems, B-I-F Industries, Providence, R.I.

A FEW years ago, a water system usually consisted of a river intake or well, a treatment plant, and a pumping station, all at the same location. Now the situation is quite different. Scattered intakes, wells, and reservoirs are often linked to one treatment plant, which connects with elevated storage tanks and booster pumping stations throughout a city area. The system may be decentralized still more if the treatment plant and pump station supply water to outlying towns, miles from the central station.

As water systems grow and spread, so do operational problems. Formerly, all the controls were in one place; now they are widely dispersed. Telemetering was the first step toward quick, economical central coordination of controls. Then elementary remote control came into use. Automatic operation, a central supervisory control system, is the latest system for coordinating scattered pumps, storage tanks, and other components.

A supervisory control system provides operating personnel with: (1) adequate supervision of remote equipment; (2) operating information about the status of all equipment; and (3) telemetry over a common transmission link or channel. Combining the ASA definitions for supervisory control, indication, and telemetry, a supervisory system may be defined as equipment for the selective control, automatic indication, and continuous telemetering to or from remotely located units by electrical means over a relatively small number of transmission channels.

Applications and Advantages

When applied to water systems, a supervisory system furnishes: (1) central control and operation of remote pumps, regulating valves, gates, chemical feeders, fans, and lawn sprinklers; (2) central operation of automatically controlled pumps, valves, or standby generators; (3) central alarm indications of excessive bearing temperatures and room temperatures, room flooding, and position of doors; and (4) central continuous telemetering of levels, flows, pressures, voltages, and currents.

The prime benefits gained from the application of supervisory control to a water system are reduced operating expense and higher operating efficiency. Depending on the system requirements, other advantages are:

1. High dependability is assured by constant watch over remote areas and elimination of human error

2. Skilled workers are released for other duties by centralizing operational and control information

3. Systems may be expanded at low cost through initial system planning that makes additions for future growth

possible without changing overall equipment

4. System efficiency and productivity are increased by instantaneous control and indication of system operation. Demands can be satisfied without the time lag involved in uncoordinated systems

5. System failures are pinpointed immediately through constant system monitoring, thus facilitating the quick dispatch of repair or maintenance parties

6. Customer relations are improved through better service.

Choice of Transmission Link

Supervisory control is especially adaptable to control and telemetering between stations separated by more than half a mile. It is common to find supervisory control dispatching equipment in system operators' offices, maintenance quarters, city halls, fire stations, booster stations, and water treatment plants. The supervisory control receiving equipment is located in well fields, lake or river low-lift stations, booster pump stations, valve vaults, and water treatment plants.

If the dispatching center is fairly near the remote receiving station where equipment to be controlled is situated, controlling and monitoring are made relatively simple by installing, at reasonable cost, multiconductor cable between stations. Spare cable pairs must be taken into account to accommodate

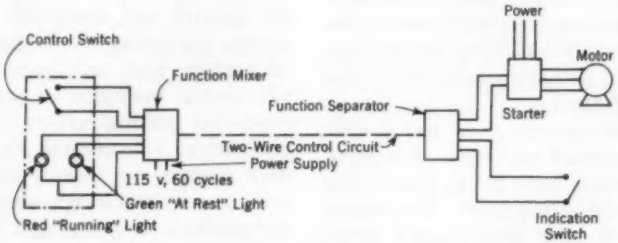

Fig. 1. Elementary Supervisory Control Unit

Control and indication (report back) are effected simultaneously over a common circuit.

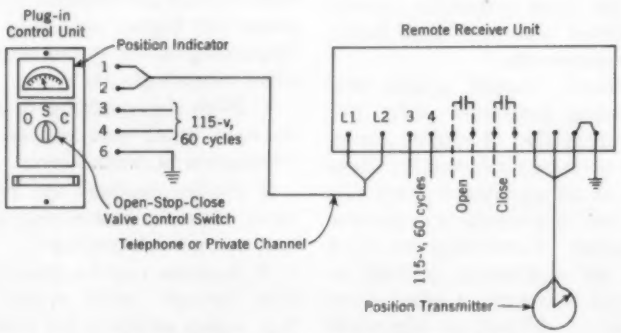

Fig. 2. Position Indicator-Controller Unit

A remote valve is nudged or positioned, and its position is indicated at the control unit.

for future additions to the system and for replacement of open or shorted pairs.

If the dispatching center is located at some distance from the receiving station, controlling and monitoring become more involved, because a long multiconductor cable is usually costly to install and maintain. In many situations, rights-of-way for new multiconductor cable cannot be obtained. In such cases, four possible transmission channels may be considered: (1) carrier wave, (2) microwave, (3) radio, and (4) leased telephone lines. The use of a leased telephone line is usually selected, because it costs the lessee nothing to maintain and appears to be more reliable for long distances, which average 15–20 mi. In the final analysis, leased lines cost less over several years.

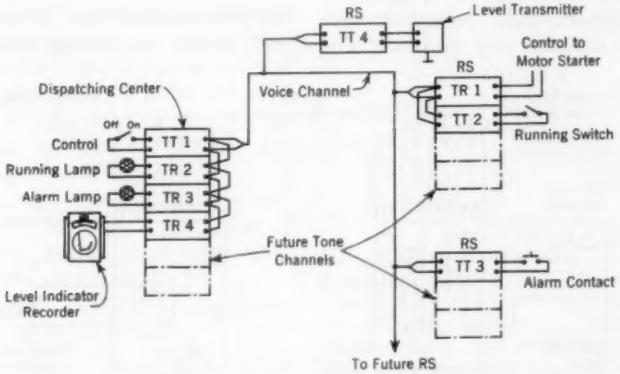

Fig. 3. Multistation System With Audio Tone Units

Audio tone frequency units over a common voice telephone channel are usually most economical for multistation systems. RS *is remote station, which may have one or more tone receivers* (TR) *or tone transmitters* (TT).

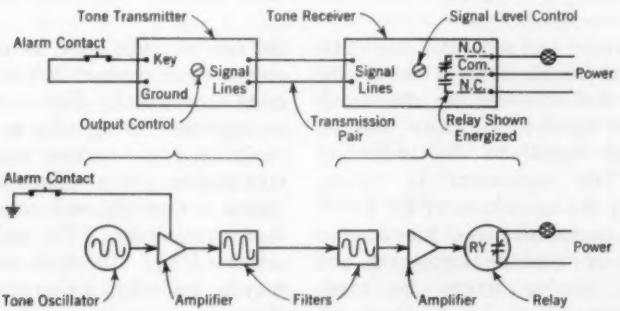

Fig. 4. External Wiring and Functions of Tone Transmitter and Receiver

Transmitter's characteristic frequency is accepted at any point along the transmission line by receiver that energizes metering, alarm, and other devices. N.O. *is normally open contact;* N.C., *normally closed contact;* Com., *common terminal between* N.O. *and* N.C.; RY, *company designation of relay type.*

Typical Installations

The following text and figures explain how supervisory control equipment is used in applications ranging in complexity from two functions at a remote well to multiple functions at a treatment or booster pump station.

Two-function control-indicator units. An elementary supervisory control arrangement, shown in Fig. 1, can be used for controlling a remote pump or valve and simultaneously receiving a positive "report back" indication over a common metallic pair or link. The function mixer and separator use static devices and polarized direct current for efficiently and economically channeling the control signal to the motor, and the report-back signal to the indicating lamps. The equipment is recommended by the manufacturer for simultaneously performing two functions in the same or opposite directions, and does not employ timers for time-sharing purposes.

Two-function position-indicator units. A similar control arrangement, employing static devices, is used for remote nudging, jogging, or regulating the position of a valve, gate, or variable-speed drive. The position indicator-controller is shown in Fig. 2. The control switch is usually spring loaded to return to the center "stop" position. A 1 ma, d-c, potentiometric position indicator is incorporated within this unit to show continuously the actual valve position as the operator nudges it to a new position.

Multistation systems. For multistation systems, comprising telemetering transmission and several control and alarm monitoring combinations, the use of audio tone frequency units over a voice channel link is considered most economical. Figure 3 shows an arrangement using plug-in type, rack mounted, transistorized tone transmitters and receivers. Each function requires a tone channel consisting of a tone transmitter (TT), and a tone receiver (TR). About 26 tone channels may be applied to a characteristic voice channel.

Figure 4 is an external wiring and function diagram illustrating the transmission of an alarm function by audio tone. A contact closure across the

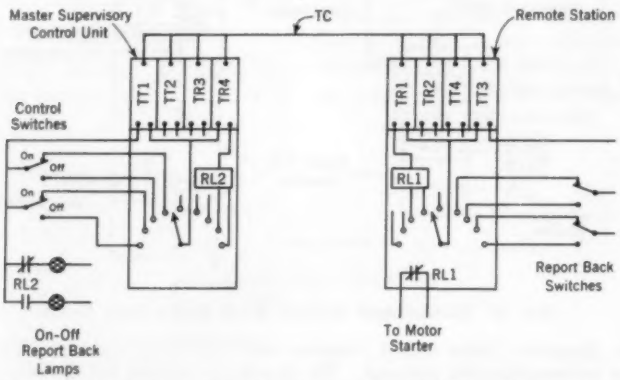

Fig. 5. Intermittent Scanning System for Multifunction Control

This system, using audio tone transmission, may have 4–26 tone channels.

"key" and "ground" terminals of a transmitter causes oscillations of its characteristic frequency to be emitted on the signal line. This frequency is accepted at any point along the signal line by an associated receiver, whose filter passes only this signal to its amplifier section in order to key a relay. Normally open or closed contacts on this relay may directly energize metering, control, alarm, or indicating devices.

Multifunction scanning systems. When multiple functions are to be handled between a dispatching center and a remote station, complete terminal equipment is furnished in appropriate free-standing cubicles, all factory assembled, wired, and tested.

Basically, all multifunction supervisory control systems employ one of the following modes of transmission, or a combination of them:

1. Pulse code, such as pulse-count, pulse-space or binary code
2. Tone code
3. Scanning-switch sequencing.

The application of an intermittent scanning system, using audio tone transmission, is illustrated in Fig. 5. This system features synchronized motor-driven scanning switches that automatically scan only when a control function is initiated from the dispatching center, or when disagreement is sensed at a remote station. Each scanning switch is rotated by a constant-speed motor at 1 rpm. During one revolution of the switches, as many as fourteen control functions may be initiated, and all report-back indications are monitored. The basic system employs four tone channels to perform 56 functions, control or advisory. Twenty-two additional tone channels are available for continuous telemetering, jogging operations, or instantaneous alarms.

Economic Considerations

Table 1 has been prepared to compare the cost incurred by converting a utility to supervisory control with the savings effected by operating with supervisory control. It represents the operational costs of a system supplying water to a community of 72,000 people and covering an area of 38 sq mi. The cost of conversion has been estimated at $121,000, with a payout period of 28 months. As itemized in the table, yearly savings for this utility with supervisory control would be $52,528. According to these calculations, the supervisory control system would pay for itself in 22 months or less.

TABLE 1

*Cost Comparisons for a Utility * With and Without Supervisory Control*

Type of Expenditure	Annual Cost—$	
	Without Control	With Control
Operating expense and overhead	77,000	22,000
Mobile equipment	1,600	0
Maintenance	8,200	3,500
Expense for expansion	7,200	0
Amortization of control equipment	0	9,000
Interest	0	6,000
Telephone line rental	0	972
Total	94,000	41,472

* Ten each of tanks, pumps, and valves.

Conclusion

Various types of equipment are available to perform the operations outlined in this discussion. Each has been thoroughly field-tested on actual installations over several years. The economy and system performance accomplished with supervisory control have been demonstrated.

System Telemetering and Automation at Madison

Nate Miller

A paper presented on Sep. 29, 1960, at the Wisconsin Section Meeting, Madison, Wis., by Nate Miller, Electrical Engr., Dept. of Water Works, Madison, Wis.

AS water systems develop, the need for operational improvements becomes increasingly apparent. Systems that were formerly operated satisfactorily over short distances are now characterized by operational difficulties that are multiplied by the addition of new pumping stations and facilities at remote locations. Modern equipment and technology are furnishing the solutions to these problems.

Central Supervisory Control

A central supervisory control system provides operating personnel located at a main station, city hall, or other headquarters with telemetered information of system variables, with remote control, and with alarm monitoring of remote pumping stations. Transmission links that can be incorporated in a system are carrier, radio, microwave, or private or leased telephone line. The most common and economical transmission link used with most water systems is the leased telephone line.

Madison, Wis., has an unusual system of central supervisory control, telemetering, and alarm monitoring of a number of remote, unattended pumping stations. The system, designed and built by the city, uses leased telephone wires. It was started in 1952 and, at that time, measured the water level in one of the ground surface reservoirs at one of Madison's remote pumping stations. The system also included alarm signals to warn against low water levels or pump shutdowns. The equipment has been expanded and now provides information from nine of the city's remote, unattended, unit well pumping stations and from three of its low-service wells.

The pumps at the three low-service wells can be started or stopped by remote control from the main station. Report-back signals verify that the pumps are responding properly. A further addition to the system is remote supervisory control of three of the city's newer, remote, unit well pumping stations that can be started or stopped from the main station. A typical pumping station, 6 mi from the main station, has a 250-hp deep well motor pumping at the rate of 3 mgd into an adjoining reservoir. The station is also equipped with a 150-hp high-service booster pump and motor. When this station is started by remote control, automatic operation of such auxiliaries as a chlorinator, fluorinator, flow meter, and an air conditioner results. Protective devices stop the booster pump when it does not provide suction. Telemetered readings of pressure and ground storage reservoir water level are transmitted to the main station. In addition, alarm monitoring of low water level in reservoirs and pump shutdown

is accomplished with the same pair of telephone wires.

Supervisory Control Unit

The left-hand cabinet of the supervisory control unit (Fig. 1) at the main station is used to control the remote unit well pumping stations with the pressure indicator in the right-hand cabinet. The equipment includes a scanning arrangement; each of the nine stations in succession is "on test" for a period of 20 sec to monitor for alarm conditions and to obtain momentary readings of pressure or water level. Pressure or water level readings are obtained on alternate scanning cycles. Alarm signals are identified on the water level indicator and by audible alarm. A green on-test light indicates the station involved. The top row of in-service red lights are turned on or off manually, with the switches directly below them, to give an indication of the equipment in operation. The switches are also used to silence the alarm. The lights directly above and those below the numbers refer to the same pumping station. A station is started or stopped when the 20-sec automatic scanning operation is changed to a time signal directed to the station involved. A continuous 1.5-min signal starts or stops a station. The left-hand dial initiates the time signal; the right-hand dial selects the proper station.

Fig. 1. Central Station Supervisory Control Unit

The control unit is kept at the attended main station. The cabinet at the left is used to control the remote unit well pumping stations; the cabinet at the right controls three low-service wells, identified by the letters E, D, *and* L. *The pointer in the gage at the left indicates "water level," "low level," or "shutdown." A pressure gage is at the right.*

The right-hand cabinet of the supervisory control unit is used to control three low-service wells (Fig. 2). These supply water to Madison's high-service pumps at the main station through a gravity flow main from a distance up to 1 mi away. The letters *E, D,* and *L* on the cabinet (Fig. 1)

identify the city's East Well, Dayton Street Well, and Low-Service Reservoir Well. The top row of red lights shows the wells in service; the lower, yellow lights are report-back or in-agreement lights. The last row of switches is used to start or stop the motors. All indications with this unit are continuous, and do not involve a scanning arrangement.

the equipment also includes devices that give alarm signals to warn against low water level and pump shutdown. Additional alarms could be incorporated in the equipment.

A pressure gage device is used as a water level or pressure transmitter. The water level transmitters first installed consisted of a modified float mechanism. These transmitters have

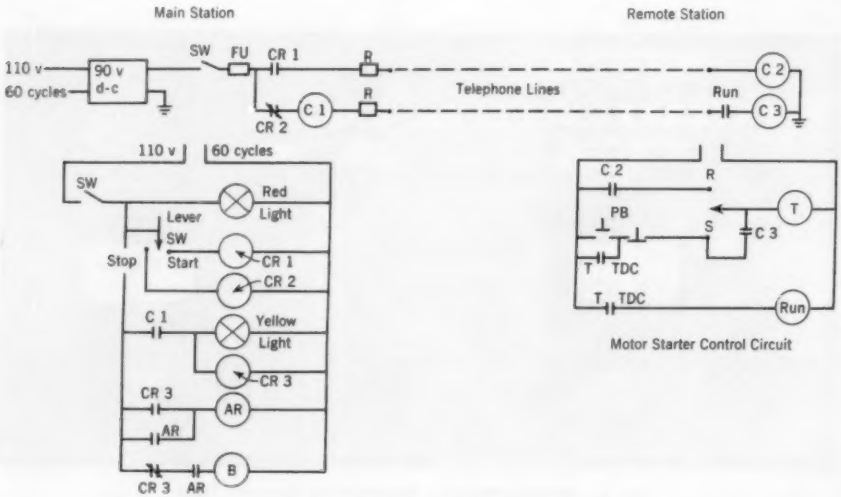

Fig. 2. Control Units for Low-Service Wells

Letter symbols in the diagram represent: AR, *alarm relay;* B, *bell alarm;* C (*1, 2, and 3*), *current relay;* CR (*1, 2, and 3*), *control relay;* FU, *fuse;* PB, *start-stop push button;* R, *resistor;* RS, *remote-station switch;* Run, *running contactor;* T, *timing device; and* TDC, *time-delay close.*

Remote Units

Remote-control equipment used for complete automation of the remote pumping stations (unit wells only) is shown in Fig. 3. With this equipment, located at unattended stations, a station can be put into operation or stopped, and information on water level and pressure can be transmitted to the main station. As mentioned before, been in use for more than 8 years and have required almost no repair. A mercury tube in the transmitter sets off the low-level alarm.

Other Equipment

Additional telemetering equipment at Madison consists of a commercial water level recorder at the main station to provide information about the

city's distant high-service, 6-mil gal reservoir. An electrode relay and hanging electrodes have been installed at the reservoir. They are used over the same circuit as that for the water level recorder to provide a bell alarm at the main station to warn against dangerously high water levels. Over the years, reservoir overflow occurred on two occasions as a result of the failure of an altitude valve. Property damage was considerable. The alarm has prevented a recurrence of overflow of this reservoir.

A high area in the city with an elevated tank is supplied with water by two booster pumps at a station about 1 mi from the elevated tank. Originally, these pumps were automatically operated by means of pressure controllers located at the booster pumping station, but pump performance was unsatisfactory. The relationship between the pressure at the booster station and the tank level was not positive because of fluctuating heavy loads near the booster station. Therefore, the pressure controllers were moved to the elevated tank and remote control equipment added. Today, the booster pumps are operated by remote control over leased telephone wires from the elevated tank. The tank level is now kept uniform, and service and operation have improved. Frequent starts and stops of pumps have been eliminated.

Savings and Service

During the last 10 years, Madison has increased in population by 40 per cent to 130,000 people. At the same time, pumping facilities have increased by 60 per cent, but automation has eliminated the need for additional operating personnel during the 10 years. It is estimated that automatic equipment paid for itself in 1 year and is now saving the city $10,000 per year in operating expenses. Additional savings are expected with the expansion of the automatic program. Furthermore, service to consumers has improved because operating conditions in remote areas become known more quickly. It was common for a consumer to call the water department

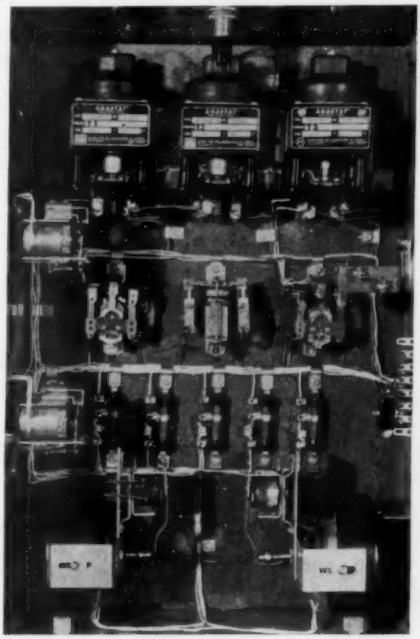

Fig. 3. Remote Telemetering Equipment

The unit at the bottom left, marked P, *is the pressure transmitter. The unit marked* WL *is the water level transmitter. Other units include a release or impulse ratchet relay and two control relays (second row) and resistors, time-delay devices, and terminals for separately mounted alarm relays. This equipment is used only at unit wells, not at low-service wells.*

when water pressure was low or when difficulties occurred in remote areas. Today, the department frequently is on the job correcting a condition by the time a consumer telephones to complain of water pressure problems.

Operation and Maintenance

Madison has experienced few operating problems with its remote-control system of automation. The remote-controlled unit wells do not need a continuous-run signal to keep them in operation during a storm that results in damage to the telephone line or in loss of electrical power at the main station. The wells are locked in operating position at the time of the telephone line failure. The position can be changed manually or when normal service is restored. The control equipment for the low-service wells depends on a continuous-run signal from the main station for continued operation. As the equipment is near the main station, it is no problem to start it manually when this is necessary.

Maintenance of automatic equipment is done by the department's electrician who is thoroughly familiar with the equipment because he took part in its construction and installation. The equipment, however, has been found to be very dependable.

Monthly Water Bond Interest Costs and Sales

A report of the Chief, Basic Data Branch, Div. of Water Supply & Pollution Control, US Public Health Service, Washington, D.C.

Month	1960–61 Net Interest Cost—*per cent*		Total Bond Sales—*$1,000,000*		
	General Obligation Bonds	Revenue Bonds	1960–61	1959–60	1958–59
Apr.	3.95	4.49	38.2	44.4	40.0
May	3.79	4.29	49.9	105.5	22.7
Jun.	3.77	3.77	11.4	50.3	20.1
Jul.	3.99	3.93	133.1	17.3	39.0
Aug.	3.34	4.02	48.4	25.5	37.7
Sep.	3.82	3.75	18.8	16.6	14.3
Oct.	3.64	3.85	21.0	68.4	60.8
Nov.	3.36	3.86	18.4	36.3	42.5
Dec.	3.55	4.20	77.3	18.4	11.5
Jan.	3.88	3.70	37.8	21.2	28.3
Feb.	3.41	3.71	19.4	41.7	62.1
Mar.	3.47	3.48	102.8	19.5	57.0
Total			576.6	465.1	436.0

Further Study of Permanent Color Comparator for Fluorides

Task Group Report

A report made under the jurisdiction of Task Group 2620 P—Fluoridation Materials and Methods, Robert S. Phillips, chairman. The project was carried out by Ray L. Derby, Robert S. Phillips, L. A. Smith, and Frank S. Taylor, under the chairmanship of Franz J. Maier, San. Engr. Director, Div. of Dental Public Health & Resources, US Public Health Service, Bethesda, Md.

A STUDY of two permanent color comparators for fluorides was published in the April 1957 issue of the JOURNAL.[1] Among the conclusions mentioned in the report was that the kit manufactured by W. A. Taylor Co., Baltimore, Md.:

> . . . although satisfactory for detecting gross fluoride feeding errors, does not produce results within the standard of accuracy defined by *Standard Methods.* Further work by the manufacturer would probably lessen this error.

Modified Testing Kit

Since the publication of the 1957 report by AWWA Task Group 2620 P—Fluoridation Materials and Methods, the manufacturer has made available a modified version of its fluoride testing kit. The modifications include a new slide containing permanent color standards in vials, longer analyzer tubes, and the use of the Scott-Sanchis reagent, which supplants the one developed by Lamar. The new tubes and slide fit and are intended to be used in the older "water analyzer" apparatus.

With the announcement of the availability of this modification, AWWA furnished the new parts and reagents to each member of the original task group, which was to evaluate the new kit. The purpose of this study was to determine, as before, the conditions that would produce optimum accuracy in the fluoride determination and also the factors that could produce results more inaccurate than those obtainable from the procedures found in *Standard Methods.*[2]

Laboratory Tests

On the basis of information obtained from the earlier study, a reexamination of the effects of reagent storage conditions and reagent life was not deemed necessary. Similarly, the effects of artificial illumination were not reinvestigated. The only points considered in this study were the effects of reaction times and sample temperatures when they varied from those prescribed by the manufacturer.

The procedure adopted was standardized for the participating laboratories. The water samples tested were those being routinely examined. In each of three successive portions, the first was run undiluted, the second was diluted one-half, and the third diluted one-fourth. The necessary volume of water for each sample was collected in one container, mixed, dechlorinated if necessary, and divided into portions

for temperature evaluation and time readings, as shown on the reporting form (Fig. 1). Those portions used for temperature evaluation were brought to, and kept at, the required temperature throughout the reaction period by means of water baths. The single portion needed for the time readings generally was the one prepared for the 25°C temperature test.

As shown on the reporting form, the samples were examined at various times (±2 min) and temperatures (±1°C). The manufacturer's recommended reaction time and temperature range are also shown on the form.

Each laboratory completed only one form, generally within 1 month's time. This represents the examination of ten samples, with ten readings made for each sample. It also includes ten additional readings by means of the *Standard Methods* technique preferred by the laboratory.

TAYLOR COLOR COMPARATOR FOR FLUORIDE

Laboratory_____

Analyst_____

(Taylor Instructions: "Exactly 1 hr," 20–30°C)

Sample	Standard Methods	Time (±2 min)					Temperature (±1°C)				
	F *mg/l*	30	45	60	75	90	15	20	25	30	35
1											
2											
3											
4											
5											
6											
7											
8											
9											
10											

Fig. 1. Form Distributed by AWWA Task Group

In the column labeled "Standard Methods," the concentration of fluoride determined with the procedure from Standard Methods *was recorded. The concentrations obtained with the Taylor kit were recorded in the other columns.*

In the calculation of the average error of the readings, the gross error for each analysis was computed by obtaining the difference between the *Standard Methods* results and those obtained with the kit. The data plotted in Fig. 2 and 3 are arithmetical averages (obtained from Table 1) of these deviations without respect to sign.

Results

In contrast to the earlier study, during which each kit tended to give

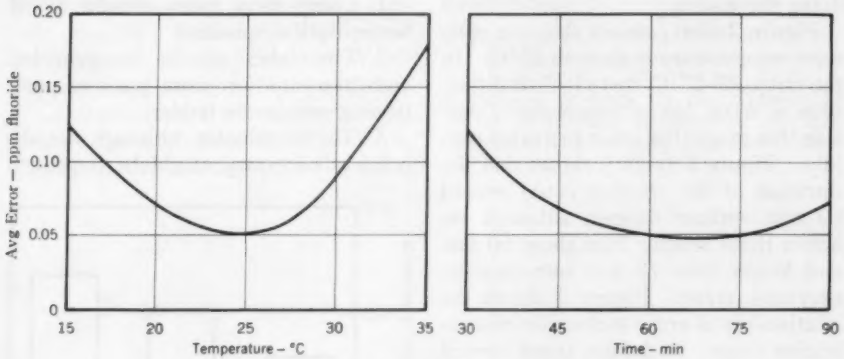

Fig. 2. Effects of Temperature and Time

The curve at the left shows that the optimum temperature is about 25°C; the curve at the right shows that the duration of the reaction could exceed 60 min.

TABLE 1

Summary of Results From Laboratories

Laboratory Number	Average Error With Time—*min*					Average Error With Temperature—*°C*					Average Error With F Range at 25°C (60 min)* *ppm*		
	30	45	60	75	90	15	20	25	30	35	0–0.40	0.41–0.70	>0.70
	Average Weighted Error—*ppm fluoride*												
1	0.08	0.02	0.05	0.07	0.10	0.11	0.06	0.05	0.08	0.12	0.025 (8)	0.100 (4)	0.050 (8)
2	0.147	0.067	0.027	0.017	0.028	0.123	0.048	0.027	0.087	0.167	0.027 (8)	0.010 (4)	0.035 (8)
3	0.060	0.045	0.020	0.045	0.055	0.125	0.040	0.025	0.050	0.155	0.008 (6)	0.033 (6)	0.025 (8)
4	0.05	0.046	0.046	0.04	0.141	0.05	0.04	0.04	0.046	0.161	0.010 (6)	0.047 (6)	0.065 (8)
5	0.06	0.05	0.07	0.07	0.12	0.19	0.12	0.06	0.06	0.31	0.033 (6)	0 (2)	0.092 (12)
6	0.26	0.18	0.09	0.045	0.040	0.08	0.08	0.09	0.17	0.27	0.050 (6)	0.083 (6)	0.125 (8)
Average (weighted)	0.11	0.07	0.05	0.05	0.08	0.11	0.07	0.05	0.08	0.18	0.026 (40)	0.051 (28)	0.067 (52)

* The number of samples is shown in parenthesis.

either consistently high or consistently low readings, the kit tested gave readings that varied from the *Standard Methods* results in both directions. For instance, when the water samples contained 1.0 ppm fluoride, readings both higher and lower than 1.0 ppm were reported by the different people doing the testing.

Figure 2 (left) shows that the optimum temperature is close to 25°C. In the range 23–27°C, there is little difference in error, but at temperatures outside this range, the error increases rapidly. Figure 2 (right) shows that the duration of the reaction could exceed 60 min without danger, although reaction times shorter than about 60 min and longer than 75 min introduce an excessive error. Figure 3 shows the relationship of error to fluoride concentration range. A definite trend toward greater error in the higher fluoride concentration range is evident, although the percentage error over the entire range remains relatively constant.

Recommendations and Conclusions

On the basis of the results obtained in this study, the instructions for using the kit might be changed to permit the reaction to continue 60–75 min, if desired, at a temperature of 23–27°C. If these specifications are followed, the average error should be less than 0.1 ppm fluoride.

This study, although limited in scope, has indicated that the modified kit is superior to the earlier model tested. In contrast to the relative inaccuracies shown by the earlier model, the modified kit should produce results approaching the standard of accuracy suggested in *Standard Methods,* when the manufacturer's recommendations are followed. Better accuracy should be attained by: (1) a more precise definition of the conditions of operation and (2) making the intervals between successive fluoride concentrations on the color standards slides closer together—namely, 0.1-ppm intervals rather than 0.2-ppm intervals. Additional improvements that might be considered include:

1. Comparison tubes should be of better optical qualities.
2. The tubes should be provided with a means for more positive positioning within the holder.
3. The illuminator, although not absolutely necessary, could be improved by a more rigid support for the light holder.
4. A spacer could be provided for maintaining a fixed distance between the light source and the tubes.

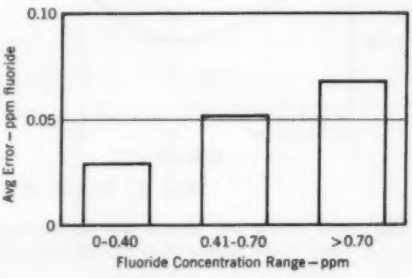

Fig. 3. Relationship of Error to Fluoride Concentration Range

Greater error was found in the higher concentration range, although the percentage error over the entire range was relatively constant.

Acknowledgments

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chemist; Ervin Hindin, sanitary chemist at the Division of Industrial Research Laboratory, Washington State University, Pullman, Wash.; F. J. Maier, chairman of Task Group 2620 P, at the USPHS Dental Public Health Laboratory, Bethesda, Md. (using Washington Suburban Sanitary Commission water), with Ervin Bellack, chemist; Robert S. Phillips, at the laboratory of the Charlotte Water Department, Charlotte, N.C., with Lee S. Dukes, sanitary engineer, and Angus Benton, chemist; Frank S. Taylor, at the laboratory of the Lake Hefner Water Treatment Plant, Oklahoma City, Okla., with Charles T. Gordon, chemist; and Harry W. Tracy, at the laboratory of the San Francisco Water Department, Milbrae, Calif., with Harold J. Lewis and Matthew J. Nunan Jr., chemists.

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Detergents and Other Contaminants in Water Supplies—1960

James C. Vaughn

A contribution to the Journal by James C. Vaughn, Chief Filtration Engr., South District Filtration Plant, Chicago, Ill.

IN the last 12 years, detergents in water supplies have become a nationwide problem. They now represent over 76 per cent of the combined soap and detergent market. The use of detergents has more or less stabilized at this level, fluctuating according to the promotional efforts of their manufacturers. The major change in the market situation is that the "non-ionic" detergents have increased their share of the market from 10 to 26 per cent in the last 2 years.

The nature of detergents has been sufficiently and frequently explained. To review it briefly, the packaged detergent used domestically contains essentially a phosphate "builder," some minor ingredients, and an emulsifying agent, usually an alkyl benzene sulfonate (ABS). Soap contains less builder and more emulsifying agent, because the agent that is the sodium or potassium salt of a fatty acid is less efficient than ABS. Actually, the chemical composition of the different packaged detergents is similar, varying only in the relative amounts of the ingredients. The composition of the detergents offered the consuming public is constantly changing. The trend has been to liquid and nonfoaming products. The household appliance manufacturers have experienced considerable difficulty from detergent foaming and have done a great deal to induce the housewife to use nonfoaming detergents. A recent summary of these trends was presented by Foster D. and Cornelius T. Snell.[1]

As the surface-active agents are divided into three classes according to their ionization in water as anionic, cationic, or nonionic, it would seem that the anionic surfactants such as ABS, constituting a large part of the domestic market, should merit most attention. It was mentioned above that the non-ionic surfactants now constitute 26 per cent of the market. Henry C. Speel has pointed out that the industrial use of surfactants is growing faster than domestic use.[2] As most of these are cationic, some attention should be given to problems arising from this source.

Toxicity

An AWWA task group report[3] states: "As a class, the cationics are the most toxic of the surfactants." Fortunately, the cationics are neutralized by the anionics, more of which are always used, but this does not eliminate the danger of accidental discharge into water supplies of industrial wastes containing a large amount of the cationic detergents that are primarily used in industry.

Stokinger and Woodward[4] classify anionics as mildly toxic. Schwartz, Perry and Berch[5] state: ". . . while some of the difficulty is due to the more efficient emulsification properties

of the surfactants—there is no doubt that dermatitis of the hands is on the increase since the advent of surfactants. Newer products attempt to correct this."

Two points of view on the same situation are of interest. In an Association of American Soap and Glycerine Producers committee report [6] this statement appears: "Cups washed in common household detergents and drained without rinsing imparted 0.2–1.0 ppm of surface-active agent to the water used to refill them. This is no more than is found in some water supplies and is below the level of detection by taste. Rinsing of the cups before they were refilled resulted in less than measurable concentration of surface-active agents." Newell and Almquist [7] state: "In washing a glass with a commonly used household detergent, rinsing it in a normal manner and allowing it to draindry, the glass was found to contain 0.195 mg of anionic detergent, which would give a concentration of 0.78 ppm if the glass were filled with water."

Foaming

Foaming at sewage plants has been reported from time to time. It occurred once at the Hyperion Sewage Treatment Plant at Los Angeles (Fig. 1). This condition has largely been brought under control by the use of water sprays, although at times it is necessary to add some defoaming agent to the water.

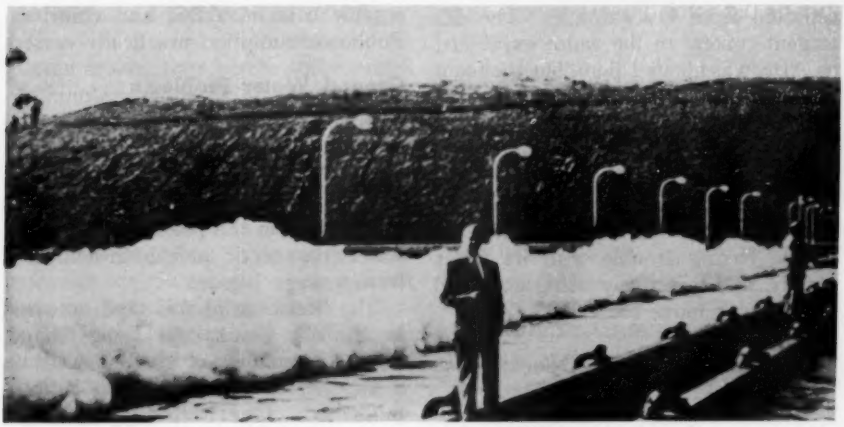

Fig. 1. Foam on Aeration Tank

Photo was taken in 1951 at Hyperion Sewage Treatment Plant, Los Angeles.

In January and February 1953, trouble was experienced at Osawatomie, Kan., a town of 4,500 people on the Marais Des Cygnes river, the source of its water supply.

The story is now a familiar one: a long period of low flows in the source river, which received a sewage plant effluent high in detergent content from a city upstream. The raw water was clear and highly colored, and had a strong fishy odor. At the treatment plant, the basins were covered with

foam. The floc, when formed, would not settle. The filtered water was colored and had a strong, disagreeable, fishy taste and a faint odor.

In December 1953, a similar situation developed at Wheeling, W.Va. The problem was much the same: a long season of low flows, and cities up river with hard water supplies (which increases the use of detergents). The characteristics of the water were much the same as those reported from Osawatomie. The detergent content of the water expressed as ABS was about 1 ppm, but the foam on the top of the water showed as much as 1,200 ppm. Spectrophotometric examination left little doubt that the Wheeling foam contained alkyl aryl sulfonate.

In March 1956, Russell, Kan., reported having trouble with its water supply, and effectively using activated carbon to remove the ABS.

Effect on Septic Tank Operation

In both England and the United States, considerable investigation has been made into the effect of detergents on septic tank sewage digestion. No evidence has been found in either country indicating an undesirable effect on this process. As is typical of most primary sedimentation and digestion sewage processes, however, the detergent content is only partially reduced by the biologic action in the septic tank.

In 1957, a situation arose at Chanute, Kan., that shows the ultimate extreme to which communities may be forced in periods of severe water shortage. Chanute takes its water from the Neosho River.[8] In the fall and winter of 1956, the flow of water in the river dwindled to practically nothing. The city officials decided to recirculate the sewage plant effluent back through the storage reservoir that served the water plant. Every precaution was taken to disinfect the water adequately. Operation of the water plant was difficult but not as serious as had been expected. As froth was a nuisance, coagulation and sedimentation were less effective. The treated water had a pale yellow color and an unpleasant musty taste and odor. It was high in organic content, ABS, and chlorides. Public consumption practically ceased.

Ground Water Problems

In recent years, since the end of the extreme drought in eastern Kansas in 1957, the detergent situation has changed in that most of the problems have arisen in the pollution of ground water from septic tank drain fields and from sewage lagoons.

The first case of this kind occurred in Suffolk County on Long Island, N.Y. A number of shallow wells in a subdivision were found to be contaminated. One well showing a yellow color was contaminated with hexavalent chromium from wastes of a plating plant. The discharge point of the plant wastes was 400 ft north of the well. A number of other shallow wells were contaminated with detergents to a greater or lesser degree.

The first move on the part of the local health department to correct this situation was to control the drilling of wells in areas containing septic tank drain fields. This was later modified when the Federal Housing Authority required that the area of a lot on which a well is to be drilled must be at least 20,000 sq ft. The authority also required the source of water supply from a water-bearing stratum to be at least 40 ft below finished ground surface

and the well to be at least 40 ft deeper than the level of the water-bearing stratum, which means that the minimum well depth must not be less than 80 ft. Wells must have watertight cases to a point at least 20 ft below finished ground surface. Furthermore, they must be at least 75 ft away from a septic tank, and 100 ft from a seepage area.

The first report of this sort of well pollution was published by Flynn, Andreoli, and Guerrera in 1958.[9]

Similar situations have been reported from points across the country. A number of wells in Ilinois have shown foaming that was attributed to detergents in ground water presumably from septic tank effluents. At Peoria, Ill., foam 1.5 ft thick was observed in one of the infiltration wells. These wells are supplied by seepage from water in a recharge pit which is filled by pumpage from the Illinois River.

At Grand Island, Neb., an interesting case occurred. Suit was brought against the city of Grand Island by one farmer through whose land the city owns a right of way for the purpose of conducting the effluent from the city's waste disposal plant to the river. The farmer asked for "permanent and mandatory injunction, general equitable relief, and costs," claiming that detergents had seeped from the city's open ditch into his well, thereby contaminating it. This suit is still pending, but it is expected that it can be settled out of court.

In Connecticut, a case has been reported by Newell and Almquist[7] in which private and industrial wells were contaminated with ABS from sewage plant lagoons. Some of the wells are as far as 800 ft from the lagoons. Newell and Almquist also reported having observed that a certain weed killer traveled as far as 105,600 ft through the ground.

In Kern County, Calif., near Bakersfield, two wells were contaminated by seepage from sewage lagoons, but a new sewage treatment system stopped foaming in the two wells. As the effluent from the sewage treatment is disposed of by irrigation, the excavation for the lagoons must have breeched the hardpan near the surface in this area and allowed seepage from the lagoons to get into the ground water.

In a survey of 2,167 shallow wells, 32.1 per cent showed the presence of ABS. As the ABS content increased from 3 to 10 ppm, the percentage of "bacteriologically unsafe" waters increased nearly 500 per cent. Other data showed that wells free from ABS were more likely to be bacteriologically safe. It was therefore concluded that ABS detergents are a useful indicator of intestinal virus contamination of drinking water from private, shallow wells where a soil absorption system is used for the disposal of domestic sewage.[10]

Methods of Removal

Experimental work at the Chicago South District Filtration Plant explored possibilities of removing the anion-active detergents in water supplies with available materials. From the results obtained, it is apparent that activated carbon is the most effective agent in removing the surfactant. Other experiments at Chicago indicated that sediment from the plant settling basins will also remove surfactants.[11]

The effectiveness of carbon in industrial plants was substantiated by the work at Russell, Kan., in 1956, although as far as the author knows,

no one has tried the settled sediment in plants. Renn and Barada [12] state that 50–89 lb of activated carbon are required to remove 1 lb of ABS when the ABS concentration is 1 ppm. Sigworth [13] states that 1–2 ppm of activated carbon for each ppm ABS will produce palatability.

Detergents can be removed from water by blowing the water with air,

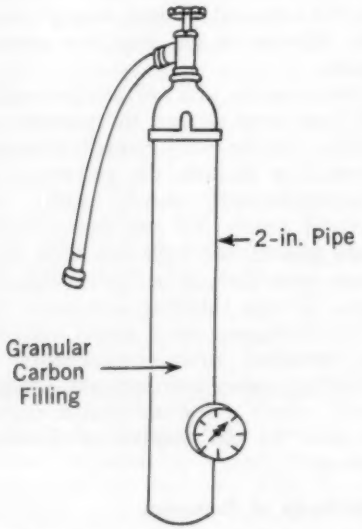

Fig. 2. Activated Carbon Column for Detergent Removal

Pipe filled with granular activated carbon was used in home experiment.

stripping off the resultant foam, and burning the foam. On a very small scale, this might be feasible, but it is difficult to imagine its application to hundreds of million gallons per day.

Because many rural and suburban homes get their water from wells, and because the problem of foaming in wells is so general, a removal method applicable to such supplies is desirable. Two solutions to this problem have been reported very recently. One is a device that consists of a section of 2-in. galvanized pipe (Fig. 2) filled with 0.5 lb of granular activated carbon. Irving M. Abrams has reported to the American Chemical Society that effective removal of detergents from a water supply could be obtained by passing the water through a column of a special resin.

The problems have been outlined and some solutions have been attempted. In general, a great deal of effort has been put into seeking solutions to problems created by detergents.

Current Action

The English established a standing technical committee on synthetic detergents in 1953, and its third progress report has recently been received. In the same year AWWA established Task Group 2661 P—Effects of Synthetic Detergents. This task group's first progress report was published in 1954.[8] Task Group 2662 P—Analytical Methods for Determining Synthetic Detergents has sponsored a project with National Institutes of Health funds to seek a better analytical method. The final report on this project is not yet available, although the work of one generally satisfactory method known as the methylene-blue process, and of another method known as the infrared process, have been published.[14] Both of these methods have been adopted as tentative and appear in *Standard Methods*.[15]

Perhaps the most extensive efforts to solve the problems created by detergents have been made by the manufacturers of detergents. For a number of years, the Association of American Soap and Glycerine Producers has had an annual budget of over $140,000 for solving the problems of ABS pollution.

In addition to the association work, the individual manufacturers have carried on extensive investigations in their own laboratories.[16]

Stream Monitoring

There have been three reports made on stream monitoring for ABS.[17-19] USPHS has established a water quality network consisting of approximately 50 sampling points widely distributed over the United States where samples of water are taken for complete analysis, including determination of ABS.

It is possible that the ultimate solution to the problems created by the presence of detergents in water supplies will be their elimination. The burden of solutions to these problems, except in periods of extreme drought, as in Chanute, Kan., has fallen upon the sewage and waste treatment plants. It has been shown that the most effective removal of ABS by such plants is approximately 50 per cent. Surfactant materials derived from sugar, and those produced by at least two petroleum refinery companies, can be completely removed from water by the biological oxidation process in the normal sewage treatment plant. It remains to be seen what portion of the detergent market these materials will capture. Perhaps the most significant trend in the solution of these problems is the use of the nonionic surfactants, which now hold 26 per cent of the total surfactant market. These materials will foam (Fig. 3), and produce taste and odor, but according to Manganelli, they are completely removed by biological oxidation in sewage treatment plant processes.[20] Unfortunately, at this time there are no analytical methods for determining the amount of nonionic surfactants in water.

The British standing technical committee on synthetic detergents undertook an experiment in the community of Luton with the cooperation of the detergent manufacturers. They supplied the housewives of the community with packages of household detergent made with a surfactant material that was completely oxidizable in the sewage treatment plant. Complete cooperation of the housewives was not

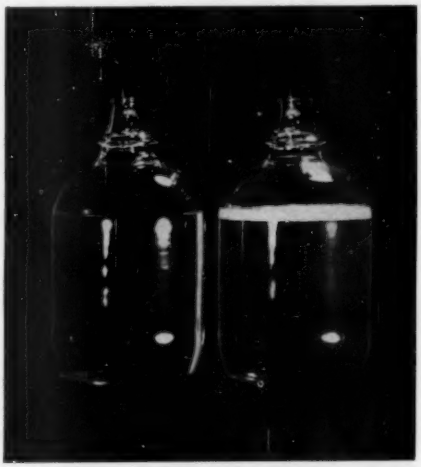

Fig. 3. Foam Caused by Non-Ionic Detergent

Bottle at left contains raw water from Lake Michigan. Bottle at right contains water from the same source with 4 ppm non-ionic detergent added.

obtained, but the majority went along with the experiment, and ABS pollution was reduced. The results were quite impressive. This is evidence that the best solution to the problem will be the elimination of ABS.

Other Contaminants

As the complexity of daily life has increased and as the chemical industry has developed, the problems of con-

taminants other than detergents in water supplies have multiplied proportionately. The effect on algae growth of phosphate leachings from irrigated crop lands into nearby streams and lakes is widely known. Excessive algae growth has on several occasions created high concentrations of a toxic material that destroyed fish life. Contamination of lakes and streams from either spills or careless dusting procedures with poisonous chemicals has damaged fish life and has been reported in the daily press. More recently attention has been called to other contaminants being produced by the chemical industry. A report by Omar C. Hopkins and Oscar Gullans [21] has called particular attention to these contaminants and to the very strict requirements as to their maximum permissible content in drinking water.

Some of the individual contaminants are:

Arsenic. This is an important element in many pesticides. Its potential health hazard and its toxicity to fish justify the recommended maximum limit of 0.01 ppm in a potable water supply.

Lead. For a number of years, the maximum permissible lead content in drinking water has been 0.1 ppm. The mandatory limit in drinking water recommended by the USPHS Advisory Committee on Drinking Water Standards is 0.05 ppm. The reason for this is that the public's intake of lead from other sources has increased greatly. The danger of lead intake from fruit sprays is always present. An interesting potential source of lead is the increasing use of small boats with outboard motors. New high-powered motors operate on gasoline containing lead tetraethyl. Outboard motor exhaust fumes, which pass into the water from the propeller shaft, leave lead and oil waste products, plus some leakage of unburned materials, in the water. Most of the lakes and streams used for recreational purposes are also used for water supply. Running motor boats on lakes and streams that are also water supplies is too well established to be stopped, but one petroleum refining company offers a high-octane gasoline that contains no lead. Other refineries must be encouraged to make such gasoline generally available to outboard motor users.

Cadmium. This is a metal of high toxicity potential. It is used as a coating on bolts, ladders, and other steel structures to prevent corrosion. At the Chicago South District Filtration Plant, no cadmium-covered material is permitted where it will come in contact with the water.

Chromium. The toxicity of hexavalent chromium has long been known, and the removal of this element from chromium-plating plant waste is generally practiced, but there is always the danger of accidental spills or poor regulation of controls. Wherever chromium-plating plants exist and their waste goes into a water supply, there is always danger of chromium contamination.

Cyanides. Although cyanides are reduced under neutral or alkaline conditions by chlorination, the mandatory limit of 0.2 ppm cyanide is necessary to protect the water users. Most large plating works have their cyanide wastes well under control, but some of the smaller installations have little or no control. Two fish kills attributed to cyanides from small plating works were recently reported in Illinois. In one of these, the cyanide content of the stream was reported as 3 ppm.

Radionucleides. With the advent of the nuclear age, the problem of radioactivity in water supplies has become acute. Nuclear testing, power reactors, and hospital use are sources of radioactivity pollution. Chicago has monitored its water supply for radioactivity for over 10 years.[22]

Selenium. With extensive industrial uses, selenium has a high potential toxicity. In the absence of sulfur in soil, wheat has been known to assimilate selenium. Treating the soil with gypsum (calcium sulfate) corrected this condition by making sulfur available to the wheat.

Vanadium is being used more and more in the steel, petroleum, and electronics industries. It is also toxic; the limits on its content in water are very low. Recent investigations have shown that chromium and vanadium are important elements in the development of cancer. This is in addition to their known chemical toxicity.

Organic contaminants. At the R. A. Taft Sanitary Engineering Center at Cincinnati, a technique for detecting organic contaminants in water has been developed. The water being tested is passed through a filter containing granular activated carbon. After the required amount of water has been filtered, the carbon is removed, dried, and washed with chloroform and alcohol. The chloroform- and alcohol-soluble portions are then subjected to chemical and physical analyses. This technique shows great promise as an information tool in the control of pollution.

Insecticides. Insecticides often have chlorinated hydrocarbons as their principal ingredient. These have a high degree of toxicity, and as reported above, have been known to travel as far as 20 mi underground. A highly toxic chlorinated hydrocarbon known as orthonitrochlorobenzine traveled from St. Louis, Mo., to New Orleans, La., a distance of over 1,100 mi. It was still present in the Mississippi River at New Orleans in concentrations high enough for positive identification. Contamination of lakes and streams from insecticides by either runoff or seepage is an ever present danger.

Hepatitis virus. One form of contamination that has come to the attention of the USPHS Drinking Water Standards Committee is the infectious hepatitis virus, which can be waterborne. It is usually transmitted from an infected individual to the sewage stream of the area. In a school, for example, over 40 cases of infectious hepatitis developed in students drinking the water from an old well into which sewage had seeped from defective drains.

Phenols. Phenols and similarly constituted compounds are famous for producing a medicinal taste in water when chlorinated. A food-processing company once buried $30,000 worth of products in one week because of a medicinal taste from phenols in the water used in its manufacture.

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Activated Carbon for Treatment of Conditions Attributed to Synthetic Detergents

—E. A. Sigworth—

A paper presented on Nov. 10, 1960, at the North Carolina Section Meeting, Winston-Salem, N.C., by E. A. Sigworth, Chemist & Technician, Industrial Chemical Sales Div., West Virginia Pulp & Paper Co., New York, N.Y.

IN the United States, acceptance of synthetic detergents (syndets) has resulted in their being used far more than soaps. Annual sales of syndets for domestic and industrial uses are presently reported to be 3.6 billion pounds.

The compound most widely used in household detergents, and the one largely responsible for the foaming tendency, is alkyl benzene sulfonate (ABS). This compound constitutes only a small part of the entire formulation, being in the range of 10–35 per cent of the total. The remainder of the formulation consists of builders, which may be plain water, phosphates, sulfates, or other compounds. In some syndets, essential oils are added to impart a perfume.

There is an extensive literature concerning potential problems to the water supply field from the presence of syndets in raw water supplies. To summarize the situation rather broadly, ABS can reach receiving streams because its removal within sewage treatment plants is, to say the least, incomplete.

Extent of Problem

None of the compounds commonly used in syndet formulations are considered toxic to human beings in the concentrations likely to be present in a water supply.[1] Thus the discharge of syndets into rivers or streams cannot be construed as a health hazard. The predominant consideration is the influence on water quality as it may affect aesthetic appreciation. The presence of a sufficient quantity of syndets in a water supply has, in some specific cases, resulted in problems of foam and possibly flavor.

Where such experiences have been encountered, correction of the difficulties was not obtainable with normal water treatment practices such as coagulation, disinfection, and filtration. Even though normal water purification practices will not correct the condition, the use of activated carbon in a sufficient quantity will result in successful correction. The purpose of this article is to suggest to the water supply field that corrective measures can be adopted and that the cost for correction will not be exorbitant. Based on analyses for ABS in a number of streams throughout the country, concentrations of ABS are, for the most part, well below levels of foaming or flavor difficulties. It is, therefore, quite unlikely that corrective treatment with activated carbon will be necessary, except possibly in isolated situations where water may be used and reused several times.

Ground Water Supplies

Well water supplies with foam and flavor problems that have been attributed to syndets have been reported in

many areas of the country, including New Jersey, Connecticut, Colorado, and Long Island, N.Y. For the most part, these problems arise in housing developments where individual wells are in rather close proximity to septic tanks or cesspools. In such wells, where foaming is sufficient to be objectionable, the condition can be corrected by installation of a pressure filter with granular activated carbon. In some instances, odors described as fishy, musty, and moldy have been reported in association with foaming. Such odors are not characteristic of household detergents, so their origin would be worthy of study. In any case, however, such odors would also be removed in the granular activated carbon filter at the same time that the foaming condition is being corrected by adsorption of ABS.

Surface Water Supplies

Where surface supplies are used for potable water, a treatment plant involving coagulation, sedimentation, and filtration will usually be available. With such a treatment plant, the use of powdered activated carbon requires a minor investment in feeding equipment. On the premise, therefore, that powdered activated carbon would be used in this treatment, a series of studies were conducted to determine its efficacy for rendering water acceptable from the aesthetic point of view.

Flavor Studies

Physiologists say that in the reaction called tasting, people employ at least three senses, namely, taste, smell, and touch, each of which apparently performs independently. Reactions made by the taste buds, located predominantly on the tongue, sense only salty, sweet, sour, and bitter tastes or combinations of these tastes. The sense of touch is reported to sense such feelings as oily, greasy, soapy, astringency, and others. All other sensations of water taken orally are claimed to be odors. Within this classification would be perfume-like, fishy, musty, and earthy.

Thus when a sample of water is placed in the mouth, each one of these three senses takes part in giving a combination of reactions which is referred to, in this article, as "flavor." These studies involved using flavor tests to determine the minimum acceptable concentrations and the carbon dosages necessary to reduce higher concentrations to acceptable levels. For these studies five different household syndets, all of them well known and widely used, were purchased from a local store and tested individually. Samples were tested at room temperature for flavor evaluation of the dilutions of the various syndets. Reactions were predominantly soapy and, in some cases, perfume-like.

In Table 1, the second column of figures shows concentrations of each specific syndet formulation that would render water unpalatable. Results are quite variable, ranging from 8 ppm for Detergent D to 20 ppm for Detergent C.

Each of these detergents was then diluted to a uniform 25 ppm concentration, and treated with varying dosages of activated carbon to determine the amount necessary to render the water acceptable from the viewpoint of flavor. The activated carbon used was a water purification grade widely used throughout the United States. Necessary plant scale dosages are shown in the last column of figures in Table 1. These results are actually half the

dosages determined in the laboratory, and are shown because it was found that results in actual plant practice are usually twice as good as experimental results.

From the flavor viewpoint, the amount of carbon necessary for correction varies widely with the different detergent formulations. For example, under the test conditions, Detergent C was only 5 ppm above the palatability limit, compared with 17 ppm for Detergent D. In both cases, however, 50 ppm of carbon was required to obtain palatability. Conversely, Detergent B had a rather strong flavor, yet required only 20 ppm of carbon for correction.

Where pollution occurs, it is quite probable that there will be a conglomeration of many detergent formulations, so it is necessary to generalize conclusions. From the last column of Table 1, it is evident that 1–2 ppm of carbon per 1 ppm of detergent will be sufficient for flavor correction.

Concentrations used for these studies were intentionally much greater than would be anticipated in a raw water supply. In fact, it is quite doubtful that concentrations in the foreseeable future will ever be sufficiently high to reach the minimum levels for flavor detection shown in the table. As mentioned above, the predominant flavor encountered in these studies was described as soapy. It should be borne in mind that this characteristic flavor has not been indicated in any of the complaints thus far reported from the field. Furthermore, no problem would be anticipated from the essential oils in some syndets, as these would probably be removed during passage through the sewage treatment plant and in the receiving stream.

Foam Studies

On the premise that any noticeable foam in a glass of water drawn from a water tap would be aesthetically objectionable, laboratory tests were set up to simulate conditions. Water samples in a separatory funnel were permitted to drop about 2 ft into a receiving receptacle, and foam was observed. If foam developed and persisted for at least 30 sec, the sample was judged unacceptable.

TABLE 1

Foam and Flavor Studies on Synthetic Detergents

Synthetic Detergent Type	Minimum Concentration of Detergent		Carbon Dosages Necessary to Treat 25 ppm Detergent	
	To Develop Noticeable Foam *ppm*	To Render Water Unpalatable *ppm*	To Correct Foam *ppm*	To Correct Flavor *ppm*
A	10.0	17.5	34	43
B	2.0	10.0	44	20
C	500.0	20.0	0	50
D	1.0	8.0	25	50
E	5.0	15.0	30	40

Varying concentrations of the five different detergents were subjected to this test procedure to determine minimum concentrations that would be objectionable. Results are shown in the first column of figures in Table 1. With Detergent D, only 1 ppm of the formulation was required to develop a lasting foam. With three of the other detergents, higher concentrations to as much as 10 ppm were required. With Detergent C, it was necessary to use 500 ppm to develop a lasting foam. It is of interest that treating this specific sample with small dosages of activated carbon resulted in greater

foaming, so appreciably higher dosages were necessary to eliminate the foaming tendency. It is suspected that Detergent C contained a defoaming agent which was selectively adsorbed by carbon. Thus higher dosages were required to remove both compounds.

All detergent formulations except Detergent C were again prepared in dilutions of 25 ppm, and subjected to treatment with varying carbon dosages to determine the amount necessary to reduce foaming to acceptable levels. Again, as in the flavor studies, the plant dosage was calculated, and results are shown in the third column of figures in Table 1. Dosages are somewhat as might be expected with the exception of Detergent D. This grade of detergent foamed more readily than any other, and yet required less carbon for correction than any of the other three tested. Again, considering a potential conglomeration of syndets in potable water supplies, it can be generalized that 1–2 ppm of carbon per 1 ppm of syndet formulation can be expected to control foam problems.

The specific test employed in these foam studies could be too severe to be representative of conditions in actual practice. An ABS content of approximately 0.3–0.5 ppm was found sufficient to be considered objectionable. It may be that field experiences will demonstrate that higher levels, possibly in the range of 1.0–1.3 ppm of ABS, could be tolerated. Should such higher concentrations be demonstrated as acceptable, then it is logical that appreciably less carbon will be necessary for accomplishing correction of the problem. Once more, it must be realized that concentrations employed in these studies were intentionally much greater than would be anticipated in a raw water supply.

Conclusion

Objectionable foam as well as any potential flavor from syndets in potable water supplies can be controlled simultaneously with plant carbon dosages in the range of 1–2 ppm of carbon per 1 ppm of syndet formulation. Considering the potentially low concentrations of syndets likely to be involved, the cost of control with activated carbon will not be excessive, particularly in comparison with many existing industrial waste problems. Of all presently used water purification processes, activated carbon is the only tool that will assure complete success. Plants facing a potential syndet problem should have ample feeding capacity to be able to apply up to 50 ppm of activated carbon at the maximum plant capacity. This high dosage is suggested because any syndet problem would probably develop during dry weather conditions when stream flows are low and water demands are at a maximum.

Reference

1. Weaver, P. J. Household Detergents in Water and Sewage. *Wtr. & Sew. Wks.*, 107:459 (1960).

Zeta Potential and Its Application to Difficult Waters

Thomas M. Riddick

A contribution to the Journal by Thomas M. Riddick, Thomas M. Riddick & Associates, Cons. Engrs. & Chemists, New York, N.Y.

THE zeta potential (ZP) is a measure of the electrokinetic charge (expressed in millivolts) that surrounds suspended particulate matter. The charge on raw-water turbidity is predominantly electronegative, and is sufficiently strong to result in significant mutual repulsion. Thus raw-water colloids and protein colloids such as skimmed milk remain suspended. If the charge on these particles is reduced to zero, these repulsive forces are eliminated. Gentle mechanical agitation will then create numerous particle collisions instead of near misses, and the forces of adhesion, cohesion, and mechanical interlocking will result in agglomeration. The ensuing buildup in particle size then enables rapid settling in accordance with Stokes' law.

Before ZP is discussed, the nature of raw water colloids and some fundamental aspects of water treatment will be reviewed.

Reappraisal of Turbidity

All suspended matter in raw water has historically been considered as turbidity, regardless of particle size. It consists of finely divided silt and clay, and organic matter undergoing microbial decomposition. Included in the latter category are algae, plant proteins, and domestic and industrial wastes discharged to watercourses.

Tastes and odors are mostly caused by the decomposition of organic matter which releases gases, soluble by-products, and solids of colloidal dimensions to raw water. Gases may be satisfactorily removed by vigorous aeration and appropriate dosages of activated carbon. Colloid removal, however, requires reduction of ZP.

Research points to the necessity of reappraising the present concept of turbidity if a highest-quality finished water is to be produced from either polluted or unpolluted sources during periods of rank algal growths.

Turbidity is electronegative and must be regarded as consisting of two distinct and separate fractions: (1) a coarse fraction with particle diameter of 1 mm–1 μ, and (2) a fine (colloidal) fraction with particle diameter of 1 μ–10 Å.

The coarse fraction may be successfully removed from raw waters by conventional alum coagulation, but the fine fraction cannot. Its colloidal size prevents sedimentation, and its electronegative ZP (in the range of 15–25 mv) prevents agglomeration. As conventional alum or ferric floc is also electronegative in about this same millivoltage range, a mutual repulsion exists between the floc particles and the colloids, and these repelling forces prevent the colloid from making permanent contact with the floc, regardless of agitation. It is believed that raw water colloids can be effectively removed only by lowering the ZP of both the floc and the colloid to a

value of approximately zero plus or minus about 5 mv. This can be done by simultaneously employing the proper dosage of an inorganic coagulant and an appropriate organic polyelectrolyte.

Research in depth, and operation at the Waterford, N.Y., water treatment plant (which is believed to be the first designed and operated on a basis of continuously controlled ZP), demonstrate these principles. In this plant a cationic surfactant is applied about 3 min after application of the alum. This results in appropriate ZP control and in highly satisfactory removal of tastes and odors.

In this approach to the problem of colloid removal, it was first necessary to devise a simple means of viewing the colloid. As a departure from the conventionally employed electron microscopes and ultramicroscopes, a new viewing technique was devised employing the stereoscopic microscope, which will be discussed later. It is now possible to evaluate visually and qualitatively the colloid content of both raw and finished waters, using for comparison water distilled at a slow rate.

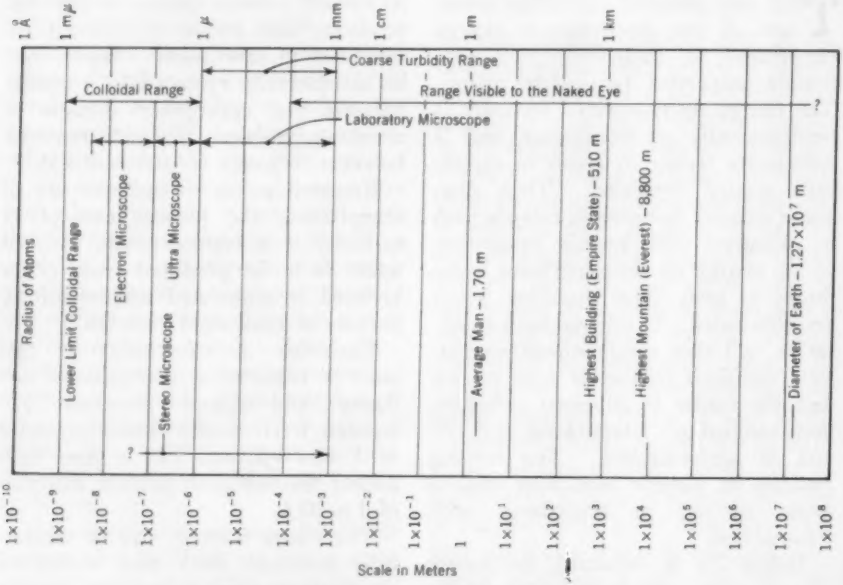

Fig. 1. Relative Size of Particulate Matter

Special illumination technique for the electrophoresis cell extends the range of the stereoscopic microscope to colloid sizes.

The colloid must be appraised in the light of both its minute diameter and its relatively large surface area. Figure 1 illustrates the size of colloids by showing the relative dimensions of particulate matter on earth. The range of the human eye is about 1×10^{-4} to above 1×10^{8} m, a spread of more than one to one trillion. The

range of the laboratory microscope is only 1–1,000. The range of suspended matter in raw water is about 1–1,000,000.

If it is assumed that typical floc has a particle diameter approximating 1 mm, then the coarsest colloid has a diameter of only 0.001 of this, and the finest colloid (invisible to even the electron microscope) has a size ratio to the floc particle of 1:1,000,000. These values emphasize the minute sizes of particles with which colloid removal must deal.

The difficulties of agglomerating and settling the finely divided particulate matter were noted above. Figure 2 shows the rate of settling of finely crushed silica particles (sp gr 2.73 at 20°C) as calculated from Stokes' law. (It is doubtful that any colloid mill could grind silica as fine as 10 Å.) The curve shows that for the upper colloid size range of 0.1 μ–1 mm sedimentation rates are only 0.03–3.4 in./day, and become negligibly slow towards the lower end of this range. Furthermore, Stokes' law deals with motion of a particle with respect to water at absolute rest. In all practical situations, however, there will be convection currents in the liquid resulting from immeasurably small temperature differences, or minor mechanical disturbances, which will be rapid as compared to these extremely slow rates of settling of colloidal particles. Hence this slow "stirring" of the water is sufficient to overwhelm completely the settling rate and move the particles up, down, or sideways, despite the forces of gravity, if the particles are small enough. The mutually repulsive force, represented by a ZP of −25 mv, is sufficient to keep each particle discrete and prevent the buildup in size required for rapid settlement.

It is noteworthy that, although highly polluted waters such as the Hudson River at Waterford contain a great deal of colloids of 0.1 μ size (with specific gravities approaching unity), appreciable precipitation does occur in less than 30 days. Whereas this would seem contrary to Stokes' law, there are indications that this

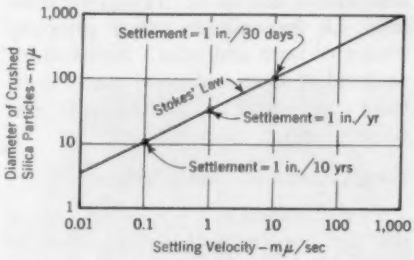

Fig. 2. Stokes' Law Applied to Suspended Silica

According to Stokes' law,

$$v = \frac{\frac{g}{\eta}(p_1 - p)d^2}{18},$$

in which v *is velocity (cm/sec),* g *is gravity (cm/sec/sec) or 980,* η *is viscosity (poises),* p_1 *is density of particles,* p *is density of fluid, and* d *is diameter of particle (cm). In this application, the fluid was distilled water with* Na_2CO_3 *added, temperature of the fluid was 20°C, sp gr of the particles was 2.73, pH was 7.2, and ZP was −25 mv.*

settlement is caused by a lowering of the ZP of the organic colloids through microbial action. Reduction of repelling forces then results in agglomeration to sizes sufficiently large to settle rapidly. This phenomenon also occurs in samples of raw water and domestic and industrial wastes that had initial ZP values of −15 to −25 mv. After several months of standing under

aerobic conditions, they are now near zero ZP, and they settle rapidly after agitation.

These observations may be summarized by stating that, whereas conventional coagulation (and often plain filtration alone) will satisfactorily remove the coarse fraction of turbidity, it does not remove the colloid. The colloid must be removed if difficult raw waters are to be treated successfully, as the colloids are a principal source of taste and odor. Satisfactory removal of taste and odor now constitutes the major (and presently unsolved) water treatment problem.

Reappraisal of Coagulation

The standard practice of relying on the jar test or visual observation of floc in the flocculator bays to establish the proper dosage of coagulant in the treatment plant should also be reappraised.

Standard practice has been to set the plant dosage at the minimum required to produce a floc of satisfactory size and settling characteristics. Such floc is invariably electronegative, and the strength of its charge is sufficient to repel the colloid. The jar test serves only to establish a floc-producing dosage, but floc produced by minimum dosages is far too electronegative to achieve colloid removal. Therefore, the jar test must be augmented by a determination of the ZP of the floc, which can only be effected by means of an electrophoresis cell. The proper coagulant dosages can be established only when the ZP's of the floc and colloids are known.

Flocculation is the most delicate of processes, and the detailing of flocculators and the selection and proper application of coagulating chemicals constitute the most critical phase of treatment plant design. With excellent flocculation, highest-quality water can be produced under conditions of heavy overload of sedimentation basins and filters. With indifferent flocculation, a raw water of really poor quality cannot be effectively treated, even if the rate of plant operation is reduced to 10 per cent of nominal design.

A fine-sized floc is normally formed when a sufficient dosage of alum has been applied to bring the ZP of the floc to about −20 mv. Upon increasing the alum dosage to that representing a carefully operated conventional treatment plant, the ZP will have been reduced to about −15 mv. Since appreciable colloid removal does not take place at this ZP, it is necessary to further reduce ZP to 0 ± 5 mv. Whereas increasing the dosage of alum normally lowers ZP, it will often be found that the alkalinity of the raw water has been severely depleted before ZP has been reduced to zero. If this occurs, unreacted aluminum sulfate will be carried through the plant, and will subsequently be precipitated in the distribution system as a result of the normal application of an alkali in the clearwater basin to inhibit corrosion. Moreover, if alkalinity is lowered to less than about 6 ppm, severe corrosion problems are created which cannot adequately be corrected by the application of lime or soda ash to the filter effluent. Polyelectrolytes, on the other hand, have the property of effecting large changes in ZP without appreciably changing either alkalinity or pH.

These conditions and limitations, as well as financial considerations, therefore dictate that coagulant dosages in the treatment plant be established as follows:

1. A jar test should be run, applying perhaps 20–100 ppm of alum in incre-

ments of 20 ppm. (In routine work at Waterford, only two dosages are normally required—for example, 40 and 60 ppm.)

2. A curve should be plotted with alum dosage as the abscissa and ZP as the ordinate. The rate of alum applications in the plant should then be set at the dosage required to bring the ZP to the range of −7 to −10 mv. This dosage, however, should be limited to that which will leave a residual alkalinity in the treated water of not less than 6–8 ppm. This is a safeguard against complete depletion of alkalinity and excessive corrosion.

3. The cationic polyelectrolyte dosage should be adjusted (in the plant or in a jar test) to further reduce the ZP to zero. Both the alum and the polyelectrolyte should be adjusted daily as required, with ZP in the flocculation basin deviating from zero by ± 5 mv at most. The polyelectrolyte dosage will normally range from 0.5 to about 4 ppm, the former representing waters of low pollution and low organic colloid content, and the latter high organic colloid loading.

Other Factors Affecting Flocculation

It should not be inferred that ZP control is a cure-all for water treatment ills, or that, by ZP control alone, floc that fulfills the overall requirements of large size, toughness, high adsorption, and quick settlement can always be produced. ZP control eliminates the repulsive forces that prevent the colloids from being enmeshed in conventional floc. Other factors that must be taken into full account are flocculator design and use and application of chemicals.

Flocculator design. There should be at least three, and preferably four, flocculation bays in series. Tangential speed of flocculator reels should be individually variable from 0.6 to 1.8 fps by means of reducers. Overall detention should not be less than 30 min, and preferably 40 min. The fundamental purpose of flocculators is not to build up large floc quickly, although this is the principal criterion to some experts. Primarily, the flocculators should effect a maximum number of impingements or collisions between the growing floc particles and the raw-

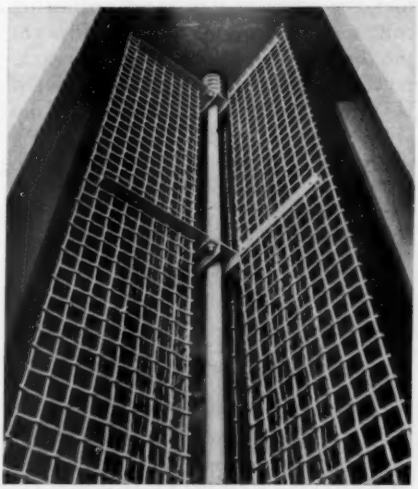

Fig. 3. Waterford Flocculator Design

Wire mesh increases the number of collisions between particles and floc.

water colloids. These collisions will result in permanent enmeshment of the colloid in the floc only if ZP is low.

It must be borne in mind that raw waters range in colloid concentration from thousands to millions of particles per milliliter. (By comparison, human blood has a concentration of approximately five billion red cells per milliliter.) At both the Wanakah, N.Y., and Waterford treatment plants, there-

fore, coarse wire or plastic mesh (patent applied for) was employed instead of the conventional redwood slats, thereby increasing these collisions many fold. Figure 3 shows this design as employed at Waterford.

Use and application of chemicals. Activated silica is necessary for coagulation of many raw waters during certain seasons of the year when the natural silica content is low. Its function is to bind together and toughen the alum (or ferric) floc complex so that floc particles will build up during continuous agitation. This is due to the formation of a long-chain inorganic polymer, and silica is unique in that it is about the only element normally present in raw water that is capable of long-chain formation. It is best applied about 1 min after the addition of the prime coagulant. A concept of polymer structure is shown in Fig. 4.

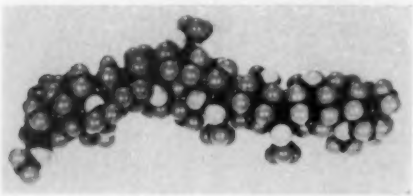

Fig. 4. Concept of Polymer Structure

The model is for methyl methacrylate.

Generally, cationic polyelectrolytes have moderately long chain lengths, with molecular weights ranging up to about 25,000. Although the floc complex is bound together and toughened to a degree by cationics, they do not supplant silica in this respect. Therefore both activated silica and cationics should be applied during the winter months if the natural silica content of the raw water is low.

The polymer should be applied approximately 3 min after the application of alum (or alum plus silica). The purpose of this time interval is to produce microflocs first, so that the polymer can become attached to them.

Stages of Flocculation

Flocculation should be considered as progressing in three distinct stages: (1) the rapid mix and microfloc stage, (2) the colloid-gathering stage, and (3) the agglomerating stage. The first stage involves the addition of alum (or ferric salts), followed by the addition of silica if needed, and then followed by the cationic polyelectrolyte. A long rectangular basin with vigorous spiral flow (patent applied for) is preferable to either the circular or rectangular basin to permit the timed and proper sequence of chemical additions. The overall detention of the first phase should be in the range of 3–5 min.

The fundamental purposes of Stage 1 are to produce tough microflocs of near-zero ZP, and to bring the ZP of the colloid to near zero. Cationic polyelectrolytes and chlorine react unfavorably. Therefore, it is imperative that the raw water contain no residual chlorine at the time of cationic application. Prechlorination may be employed, provided all chlorine is neutralized with a proper reagent prior to application of the cationic.

Stage 2 is for enmeshment of the colloid in the gradually building floc. It is the most important phase of all water treatment. This phase begins at the entrance to the first flocculator bay and extends through the third bay. Detention should approximate 30 min. Unless the colloid is enmeshed in the floc during this stage, little more can be done to prevent its passage through the plant. Here ZP control is wholly relied on to remove the mutual repulsions existing between colloids, the floc, and the floc and colloids. It

should again be stressed that merely removing these repelling forces does not produce excellent flocculation. Continuous agitation is also necessary to create as many impingements as possible between the floc and colloids. The most effective agitation is one that produces a maximum of such impingements. It is neither necessary nor desirable that floc particles be built up to large size at this stage, for small particles (for any given coagulant dosage) provide a greater total surface area. Flocculator speed should be as high as possible, in order to secure a maximum number of impingements, but not so rapid as to arrest the buildup of floc.

Stage 3 takes place in the last flocculation bay, where detention should be 7–10 min. The sole purpose of this stage is to agglomerate the floc and build it into large particles which will settle rapidly. This can best be done by introducing a polymer of very long chain and high molecular weight at the entrance to this bay. Molecular weight of the polymer should be in the range of 500,000–1,000,000 or more. Most polymers of this type (natural or synthetic) are anionic, and tend to shift the ZP slightly in the electronegative direction. The undesirable effect on ZP at this stage of flocculation, however, is more than counterbalanced by the ability of the long polymer chains to become simultaneously attached to several floc particles. This results in a mechanical as well as electrical interlocking, and produces floc of tremendous size which settles rapidly.

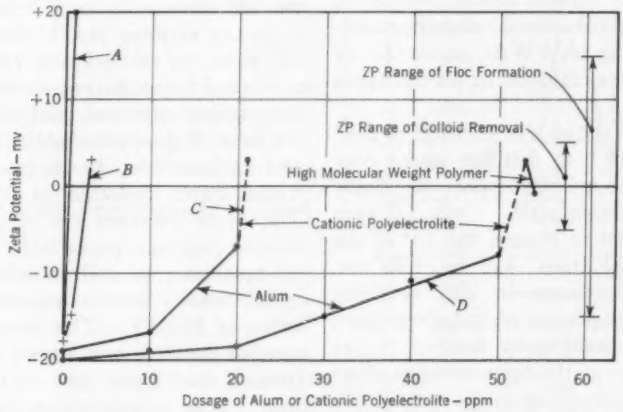

Fig. 5. Effect of Alum or Cationic Polyelectrolyte on Zeta Potential

Commercial aluminum sulfate or cationic polyelectrolytic has been added in various doses to different waters.

Weighting agents have not received due consideration in the field of coagulation. Although the rate of sedimentation varies directly with particle size, even large-sized floc may settle slowly owing to its inherent low density. This unfortunate condition can be materially improved by weighting the floc. Weighting is accomplished at both Waterford and Oneonta, N.Y., by me-

chanically enmeshing very finely divided calcite in the floc during its formation, so that each floc particle will contain about 50 discrete calcite crystals. Although calcite is electronegative, its ZP can be adequately controlled by a suitable polyelectrolyte. Barium sulfate is an even better weighting agent, as its specific gravity is 4.5, compared with 2.7 for calcite.

Zeta Potential Control

Figure 5 shows typical results of additions of commercial aluminum sulfate meeting AWWA standards or cationic polyelectrolyte to several types of water.

Curve *A* shows the addition of cationic No. 14 * to distilled water containing about 500 ppm of finely crushed Ottawa sand. Only 1.0 ppm was required to change the ZP of the silica colloid from −19 to +20 mv. This large change in ZP, resulting from a low dosage of polyelectrolyte, was made possible by the low degree of ionization of the suspending medium and by the condition of the suspensoid, which was inorganic and unreactive. The specific conductance of the distilled water was 6 μmho, and this was not appreciably increased by the addition of the silica.

Curve *B* shows the change in ZP of raw-water colloids upon addition of cationic No. 14. This was a typical New York City tap sample, which represents unpolluted reservoir water with no chemical treatment other than chlorination. Specific conductance was 125 μmho. The colloids were predominantly of inorganic origin (finely divided silt). The organic fraction (color—20 ppm) was derived from the leaching of decomposing vegetation on the watershed, and decomposing aquatic plant and microbe growths. The ZP was changed from −18 to +3 mv by 3 ppm of cationic No. 14. The cationic polyelectrolyte alone could therefore have enabled quasiautogenous coagulation.

Curve *C* shows the addition of two successive dosages of alum (10 ppm each) to this water, which changed the ZP from −19 to −7 mv. Then 1 ppm of cationic No. 14 changed the ZP from −7 to +3 mv, resulting in a marked improvement in floc formation, colloid removal, and clarity.

Curve *D* shows the addition of alum and cationic No. 14 to raw Hudson River water collected at Waterford. The river receives the discharge of sixteen pulp and paper mills within 40 mi upstream, as well as untreated domestic wastes from a watershed population of 150,000. The combined volume of these wastes varies seasonally from 2 to 13 per cent of total river flow. The concentration of organic colloids is very high, and greatly exceeds inorganic colloids. Alum in the amount of 50 ppm was required to change the ZP from −20 to −8 mv, whereas 3 ppm of cationic No. 14 changed the ZP from −8 to +3 mv. Floc formation was visually evident at 30 ppm of alum (ZP, −15 mv), slowly improving with successive alum dosages. The addition of the cationic markedly improved floc formation as well as colloid removal, as evaluated by microscopic and light-scattering techniques. After 20 min of slow-speed agitation, 0.6 ppm of a long-chain polymer of high molecular weight (anionic No. 4) was added. This

* This designates the most effective cationic polyelectrolyte of about 50 commercial brands evaluated. It is a polyamine.

greatly increased the size of floc particles and resulted in rapid settlement. As the polymer was anionic, the ZP was changed from +3 to −1 mv. It should again be noted that, whereas floc formation is visually evident at a ZP of about −20 mv, colloids are not effectively removed until the ZP has been further decreased to about −5 mv.

It is beyond the scope of this article to discuss the relationship between the character of raw water and the required alum and polyelectrolyte dosages. It should be pointed out, however, that for any given coagulant and cationic polyelectrolyte, the dosages vary principally with the organic colloid content, alkalinity, pH, and dissolved mineral salts of the raw water.

Zeta Potential Variations of Colloid Particles

It should not be inferred that the ZP of discrete colloids constituting a system such as raw water, skimmed milk, or even crushed Ottawa sand are all identical. Although such systems are usually reported as a single value (with a precision of ±1 mv on repeat determinations), this value is computed by averaging the electrophoretic velocities of, perhaps, ten or twenty particles, among which there may be considerable variation. These variations are not adequately shown by timing and individually recording twenty particles, but are clearly revealed if 100 random particles are individually timed. The electric timer employed, activated by a very fast acting switch, enables the basic data for a 100-particle count to be recorded in about 10 min. (An assistant is required to tabulate the readings.) These data plot as closely reproducible curves, examples of which are shown in Fig. 6 and 7.

The tree-type curve is well adapted to data of this sort, as it shows the relative distribution on both sides of the median. Tree-type curves of many natural colloids are often quite symmetrical about the median, with the median approximating the average. This is not invariably the case, however. Distribution curves of both

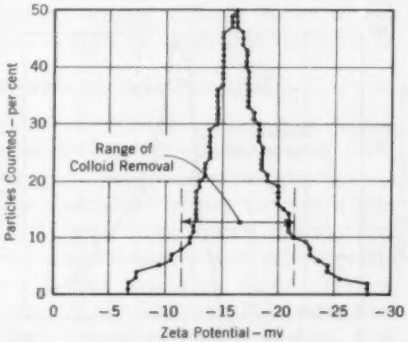

Fig. 6. Zeta Potential Distribution in Raw Water Colloids

In the sample of New York City tap water, 100 particles were counted. Median ZP was 16.2 mv; average was 16.4 mv; and ratio of median to average was 0.99:1.

skimmed milk and finely crushed silica are similar to that of the raw-water colloids shown in Fig. 6. Raw-sugar juice colloids, however, often give a steeply pitched curve from the lower ZP value to the median, and a flatter curve from the median to the maximum ZP, as is shown in Fig. 7.

The particle distribution curves are significant, for they show the range of repelling forces encountered in the colloid system, and indicate the limitations of effective colloid removal.

As stated above, the ZP of both the average colloid and the average floc particles should be brought to zero, with a working tolerance not to exceed about ±5 mv. Experience at Waterford, where more than 1,000 ZP determinations have been made, shows that Hudson River raw-water colloids are not effectively removed unless their ZP is within this range.

It is obviously impossible to bring *all* colloids to zero ZP in systems similar to the above. In Fig. 6, if the average colloid is brought to zero, then 8 per cent of the lower ZP fraction and 10 per cent of the higher ZP fraction will still lie outside the overall spread of effective removal of ±5 mv. In Fig. 7, this 10-mv spread would include all colloids on the low ZP side above 2 per cent, but would leave 14 per cent of the high ZP colloids outside the range of effective removal.

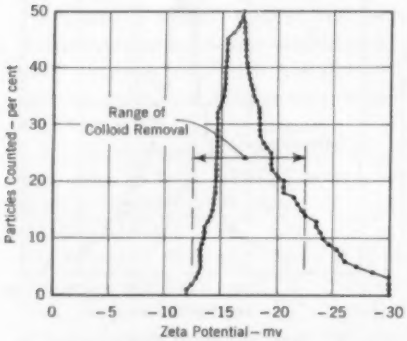

Fig. 7. Zeta Potential Distribution in Raw Cane Juice Colloids

Each dot represents one particle counted. Median ZP was 16.8 mv; average was 17.7 mv; and ratio of median to average was 0.95:1.

The best overall solution is, therefore, to apply a sufficient dosage of inorganic coagulant and polyelectrolyte to bring the average colloid to at least zero, and preferably to a positive value of perhaps 3 mv. This favors removal of the more electronegative colloids and provides some reserve for the anionic polyelectrolyte introduced in the last flocculator bay. The anionic polyelectrolyte renders the colloid more electronegative (by about 3 mv) when applied at a normal dosage of about 0.6 ppm. The colloids in the lower ZP range are readily removed, as they are all brought to or past the zero point, and, after agglomerating, they will not again disperse at positive values of about 5 mv.

The practical aspects of this natural spread in ZP values of raw-water colloids are quite apparent in either the jar test or in plant operation. If raw water is flocculated at an average ZP value (of colloids and floc) of 0 mv, most of the floc settles quite rapidly. There will still remain, however, a very small percentage of both floc and unagglomerated colloids which settle very slowly, if at all. If the ZP of this fraction is determined (represented in plant practice by water on top of the filters) it will always be found to be electronegative, often in the range of −6 to −10 mv.

This natural limitation in control of the ZP of both floc and colloids raises a doubt concerning the statement that in the future, with effective ZP control, filters may be eliminated. It is believed that filters will always be required to strain mechanically this fine-size, nonsettleable floc. According to this view, complete colloid removal can never be achieved on a practical scale, even by distillation. Control of ZP, based on the average colloid (or floc particle), however, is quite adequate

to produce a highest quality finished water from highly polluted sources.

It should be pointed out that bacterial cells, both dead and alive, have ZP's ranging from 0 mv to about −40 mv, with extremes to −80 mv or more. These cells are quite visible under effective illumination, and undoubtedly dead cells show up in the distribution curves, principally at the extremes. (Motile bacteria are identifiable if they exhibit transverse movement and, of course, should not be counted.) Good technique in routine determination of average electrophoretic mobility, therefore, dictates that neither the slowest nor the fastest moving particle be counted.

It should also be pointed out that, whereas research shows maximum colloid removal to take place at a ZP of 0 ± 5 mv, it is quite possible that for some suspensoids, the optimum may be shifted a few millivolts on either side of zero. It is also possible that the effective spread for specific suspensoids may vary from the considered values of ± 5 mv. In evaluating the fundamental accuracy of the ZP determination, the limitations of the technique and its basic formula should also be borne in mind. These points are well covered by Brinton and Lauffer.[1]

It may therefore be stated that although ZP control has its limitations in theory, technique and effectiveness, these are not of a serious nature, as ZP control does enable a very high degree of colloid removal. Present practice in the most efficiently operated water treatment plant without ZP control does not result in appreciable colloid removal, although colloids are a principal source of taste and odor.

In raw-sugar production as well as in refining, colloids constitute by far the most difficult operating problem. A few scientists and technicians in the sugar field are now becoming aware of this and of the role of ZP in precipitation. Colloids are divided into two broad categories: the lyophobic, which have little affinity for water, and the lyophilic, which have great affinity for water. ZP control is undoubtedly the key to effective precipitation of lyophobic colloids. With lyophilic colloids,[1] the additional and sometimes difficult factor of solvation must be taken into full account.

Nature of Zeta Potential

The fundamental principle of electroosmosis (the forerunner and in many respects the inverse of electrophoresis) was discovered about 150 years ago. In the early experiments, it was noted that if a ball of clay was submerged in water, and two glass tubes were inserted in the clay with their ends projecting above the water surface, then a d-c voltage connection to electrodes placed in the tubes would cause the liquid to be elevated above the water surface in one tube and depressed in the other. A reversal in polarity of the electrodes would reverse the order of liquid elevation or depression. Here was an immovable solid (the packed clay) causing a movement of the liquid. In electrophoresis, a suspended solid (such as discrete particles of colloidal clay) moves in a d-c field with relation to the suspending liquid. The present high state of development of ZP theory reflects the work of Verwey,[2] Overbeek,[3] Abramson, and others.

The basic meaning of ZP is illustrated in Fig. 8. The pictured colloidal particle is electronegative, as it has at its outer surface (or has adsorbed at

its outer surface) an excess of negative ions. In nature, practically all raw-water colloids are electronegative. Proteins (of which skimmed milk is an example) are also predominantly electronegative, with perhaps one particle in a million being electropositive. The preferential adsorption (or existence) of negative ions at the surfaces of most colloids has been the subject of much investigation. An oversimplified explanation is that these are more often OH^- ions and that they probably fit better into the lattice structure of the colloid—or are more acceptable by the colloid—than the often larger positive ions.

On this, Brinton and Laufer [4] comment:

The forces involved in preferential adsorption of ions to the surface are essentially electrostatic in nature, the ions being attracted by ionized groups, induced electric dipoles, and hydrogen bonds. In addition to binding ions, many substances can bind water; the greater the amount of water bound to the surface, the less the ion adsorption. The hydration of the ion is important for the same reason; negative ions, being less hydrated than positive ones, are more strongly bound. Another way of looking at the same question is that the affinity of water for positive ions is greater than for negative ions, so that negative ions tend to

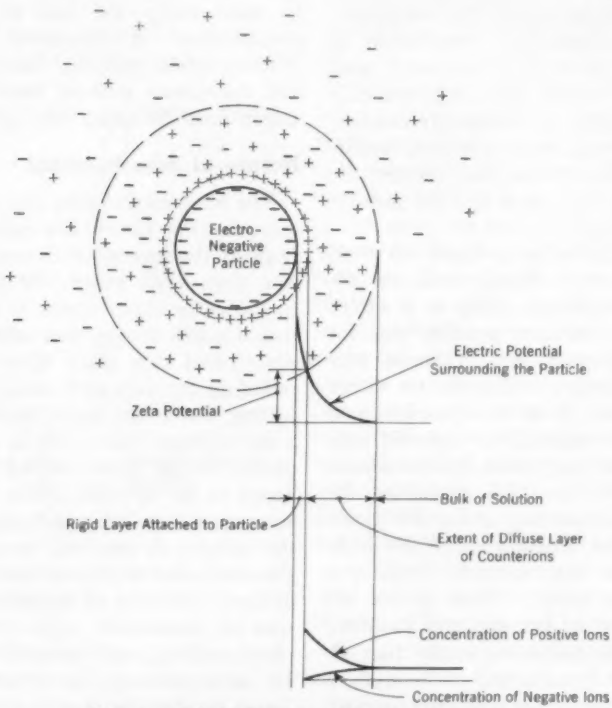

Fig. 8. Colloid Particle With Zeta Potential

The particle pictured, like nearly all colloids, is electronegative.

be excluded from the bulk of the liquid and to accumulate at interfaces. Gas bubbles, for instance, acquire a negative charge. The great polarizability and smaller size of negative ions also contribute to their preferential adsorbability.

Glasstone notes [5]:

Substances like silicon carbide, cellulose, sulfur, and carbon, which do not ionize, are negatively charged in contact with water, and the addition of small amounts of univalent electrolytes tends to increase this charge. It is probable that in these cases the negative ZP is due in the first place to the firm attachment to the surface of hydroxyl ions from the water and possibly also of anions from the electrolyte. An equivalent number of positive ions, some closely held in the fixed part of the double layer and the remainder in the diffuse portion, will be left in solution. . . . If the solid material is a substance capable of ionization, then the sign and magnitude of the ZP may be determined by such ionization. For example, silica, tungstic and stannic acids, acid dyestuffs, soaps, and glass are negatively charged to contact with water, since in each case there is a tendency for a small cation, for example, a hydrogen or alkali metal ion, to pass into solution, leaving the complex anion to remain part of the positively charged solid.

The negative surfaces of the colloid attract a surrounding layer of positive ions, which may originate either from the bulk of the suspending liquid, or from the surface of the colloid. The oppositely charged ions, or "counterions" as they are generally called, are drawn to the colloid by electrostatic attraction, while thermal agitation or Brownian motion tends to distribute them uniformly throughout the solution. This charged system—the surface of the colloid and the neutralizing counterions—is called a "double layer."

If the negative charge on the colloid is large, some counterions will be so strongly attracted that they will stick to the surface of the colloid as a firmly attached compact layer, often called the Stern layer. The Stern layer partially neutralizes the charge and electrostatic attraction of the colloid so that the remaining counterions can, on the average, be farther away while still being kept in the immediate vicinity of the colloid. They form the so-called diffuse part of the double layer, or the diffuse layer.[6]

The attractions of the central colloid is greatest, of course, close to the colloid, both because of the distance involved and because the counterions near it interpose their positive charge and thus shield those farther away. Hence, the neutralizing counterions are most concentrated near the colloid, and they become gradually negligible farther away. Similarly, negative ions of any salts present tend to be repelled from the immediate vicinity of the colloidal particles. The dimension of thickness of this diffuse layer depends primarily on the concentration of simple salts in the liquid; if their concentration is large, this dimension may be reduced to a few angstroms.[7]

The zeta potential is the potential at the surface that separates the immobile part of the double layer from the diffuse part. It is a simultaneous measure of the charge of the diffuse layer (per unit surface of the colloid) and of its extent from the colloid surface. It is therefore related to the force and distance over which the particles can repel each other and thus prevent flocculation.

This, of course, is an oversimplification. Verwey calls attention to a condition in which the surface charge on the colloid may reflect the nature

of the adsorbed capillary-active ions or the nature of the potential-determining ions (principally valence or equivalent valence) situated in the double layer.

The Schulze-Hardy rule states that in the coagulation of sols, the flocculating (hence, the ZP-controlling) power of bivalent ions is approximately 20–80 times that of univalent ions, and the flocculating power of trivalent ions is 10–100 times that of bivalent ions.

When a liquid containing such charged particles is placed in an electric field, the negative particles are attracted to the positive electrode and the counterions to the negative electrode. This attraction increases with the charge on the particle. Friction between the particle and the surrounding liquid containing the diffuse layer slows down the motion toward the electrode. The greater the extent of the diffuse layer, the lower this frictional resistance. Therefore, the velocity of the particle in a given field increases with an increase in density and extent of the diffuse layer—both of which, as explained above, are measured by ZP. Hence the velocity of a colloidal particle in an electric field is proportional to that field (volts per centimeter) and to the ZP of the particle.[8]

With this oversimplified explanation of the colloid, the double layer, the dif-

Fig. 9. Equipment for Determining Electrophoretic Mobility

This equipment will soon be available commercially.

fuse layer, and ZP, it may be worthwhile to mention the apparent consequence that the *total* charge on the colloid is directly proportional to its rate of movement in a d-c field. Abramson and others have demonstrated experimentally that such a proportionality is strictly a surface phenomenon. If, for example, a particle of calcite with a ZP of −20 mv is coated with a protein which has a ZP of −12 mv, then the protein-coated calcite will migrate in a d-c field with a velocity equivalent to a −12 mv potential.

The fundamental equation for determining the ZP of particles of a size comparable to raw water colloids is:

$$ZP = \frac{UV_t 4\pi}{D_t}$$

in which U is electrophoretic velocity of the particle, and V and D are the viscosity and dielectric constant of the suspending liquid respectively. Since V and D are constant for any given liquid at constant temperature, ZP is directly proportional to electrophoretic velocity of the particle.

Transforming voltage from electrostatic to practical volts, U is expressed as the rate of particle travel in microns per second per volt per centimeter. The microns per second are readily determined by timing the rate of travel of an individual colloid between known grid lines of an ocular micrometer. The volts per centimeter fundamentally represent the voltage drop (in the suspending liquid) between electrodes,

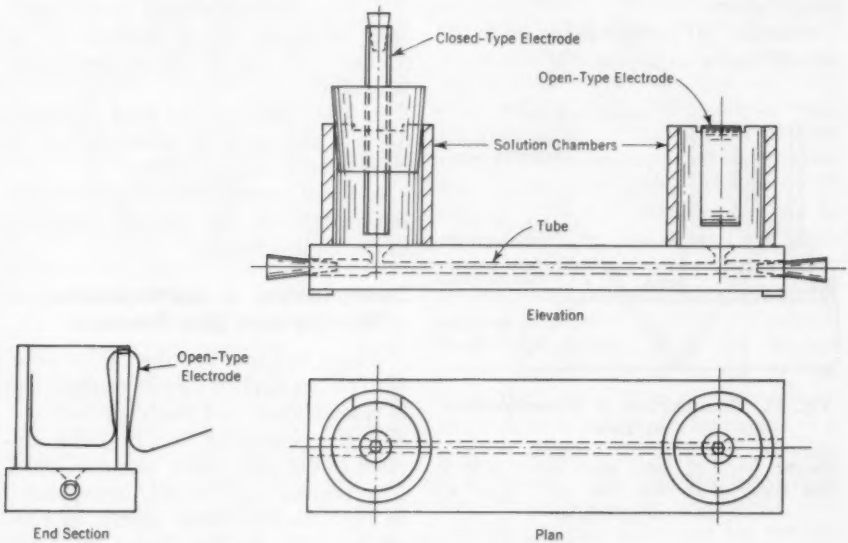

Fig. 10. Details of Electrophoresis Cell

Three views of the electrolysis cell—side, top, and end—show its design. Electrophoretic mobility, a function of ZP, is measured in tube with stereoscopic microscope.

and the effective distance in centimeters between the electrodes.

Since the turn of the century it has been known that colloidal-size lyophobic particulate matter bears an electric charge, and that this charge, either positive or negative, is responsible for its remaining suspended. It has also long been known that if the charge is reduced to about zero, agglomeration generally takes place and results in precipitation.

Whether ZP is regarded as an insignificant factor or a powerful one when

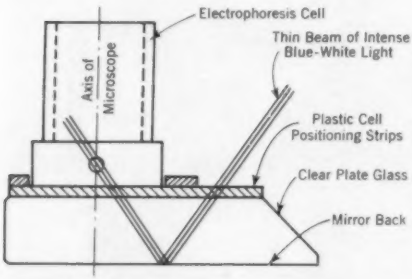

Fig. 11. Illumination of Electrophoresis Cell Tube

Mirror back of plate glass base reflects thin beam of intense blue-white light up through cell tube.

compared to such other influences as gravity depends entirely on the size and distance apart of the particles considered. Two boulders suspended side by side in a lake will show appreciable downward tendency due to gravity, and the repulsive force due to the ZP will be very small. With two particles of beach sand simultaneously dropped into the water side by side, the force of gravity is still the stronger influence. If particles are the size of 1 μ or less, however, they may remain suspended indefinitely because ZP may produce sufficient repulsive force to prevent them from coming together and agglomerating so that a buildup in size could result in sedimentation.

The reasons that such a basic principle as ZP has not been put to wide practical use by industry or by the water works profession can be enumerated:

1. Most of this research has been carried out by pure scientists who had little interest in the commercial field.
2. The electrophoresis cells presently available are so delicate as to confine their use to highly specialized laboratories.
3. The technique has been so exacting and the speed of determination so slow as to be discouraging.
4. Until recently there has been little information on the subject published in the United States.

Determination of Electrophoretic Mobility and Zeta Potential

About 10 years ago the calcite process (patent applied for) of coagulation, in which alum and finely ground calcite are prereacted before being applied to the raw water, was developed. This process was formally incorporated in the Oneonta water utility, and together with the floc feedback system resulted in the ability of this plant to operate at considerable overload. This work was followed by further research on coagulation of difficult raw waters and of synthetic waters until it was possible to form excellent floc at will in distilled water even at refrigerator temperature. This was accomplished by applying carefully chosen dosages of prereacted alum-calcite, sodium bicarbonate, and activated silica. Good flocculation in distilled water (at moderate chemical dosages) was found impossible if either the calcite or the silica was omitted.

As it had been noted in the development of the calcite process that taste and odor removal correlated quite well with the clarity of the plant effluent, research thereafter was directed principally along lines of colloid chemistry, the most promising facet of which seemed to be ZP. It was found that the cells available were not well adapted to determining electrophoretic mobility of particles even as large as fine floc. Because of their thin sections, these cells were found to be delicate and tedious to operate. It was therefore necessary to develop a new cell and a new technique. The new technique requires use of the stereoscopic, rather than the standard laboratory, microscope.

Figure 9 shows the final prototype of this equipment, which will soon be commercially available. The equipment consists of:

1. A stereoscopic microscope with special ocular micrometer; 15× eyepieces; 2×, 4×, 6×, and 8× (adjusted magnification) objectives; and a special mechanical stage (A standard compound laboratory microscope cannot be employed with the new cell.)
2. A special illuminator capable of producing a thin beam of intense blue-white light, with a heat-absorbing filter
3. A d-c power supply, continuously variable from 0 to 500 v
4. A clear plastic electrophoresis cell equipped with platinum-iridium electrodes
5. A cell holder consisting of a thick and highly reflective mirror for reflecting the light 45 deg upward through the tube of the cell
6. An interrupted-type cumulative-reading electrical timer, reading in seconds and tenths of seconds.

Figure 10 shows the cell, which consists of a clear plastic block with a small polished tube extending through the center. The tube is connected by tapered ports to two solution chambers. One chamber is sealed with a direct-coupled platinum-iridium electrode of the closed type; the other contains an open-type electrode.

The closed-type electrode is positioned in one solution chamber only, and prevents all movements within the cell tube other than that induced by the impressed voltage. The other solution chamber is left open (or covered with a loose-fitting plastic cap) for convenience in use. This arrangement is normally employed for measuring the electrophoretic velocity of all colloids. If floc particles are to be evaluated, then two open-type platinum electrodes are preferable, as they permit tilting the cell back and forth until a floc particle is properly positioned for timing. The cell is supported on a holder as shown on Fig. 11 so that a thin beam of intense blue-white light is reflected (by means of a mirror) upward through the cell tube at an angle. This removes direct light from the optics of the microscope, and permits particles too small for normal observation to be readily "seen" if they reflect light upward through the microscope. In this respect the illumination of particles is similar to the ultramicroscope and to the principle of the heliograph. The cell is positioned somewhat above the top of the mirror to provide an air-gap heat barrier. Passage of the light through the thick mirror further reduces heat transmission.

The rate of migration (electrophoretic mobility) of *B. subtilis* can be readily ascertained with a 4× or 6× objective and 15× eyepieces, al-

though the length of this organism is only a few microns. Thus this reflected light technique, with 60× or 80× magnification, enables the investigator to "see" minute particles that would otherwise require 1,000× to 1,500× magnification.

This principle of reflected light can be applied to viewing colloids in any liquid, provided the concentration of particles is not excessive. Both raw cane sugar juice and blood require dilution with distilled water. This technique is simple but highly effective in qualitatively appraising colloid concentrations, and would seem to have application in industrial fields.

Figure 12 shows a section of the tube as it is seen through the stereoscopic microscope. The 4×, 6×,

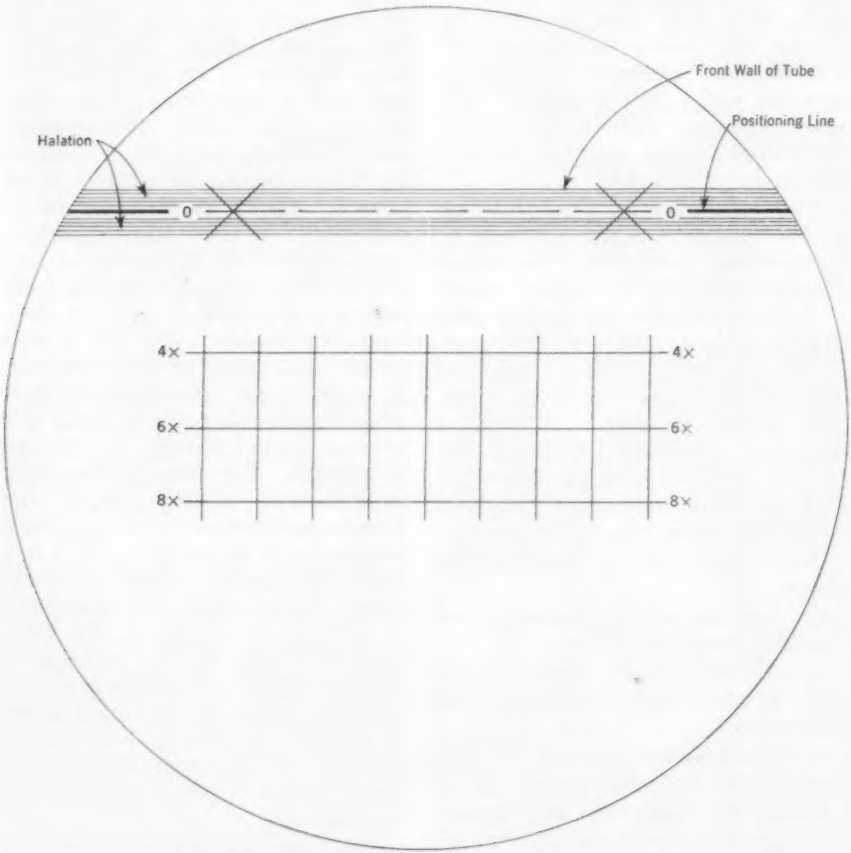

Fig. 12. Position of Cell Tube and Ocular Micrometer

A section of the tube is seen through the stereoscopic microscope with an ocular micrometer in one eyepiece. The ocular micrometer is specially designed for the cell.

and 8× objectives are preferable for precise work. The 2× objective permits viewing the entire width of the tube, and may be used for calibration. A special ocular micrometer is used in one eyepiece, and the cell is positioned with the zero micrometer line set at the apparent, not actual, front wall of the tube. The front wall is clearly visible as a long, sharp line when the microscope is focused to exact mid-depth of the cell.

Electrophoresis, or electrophoretic mobility, is the movement of a suspended particle through the suspending liquid. If a suspended solid particle moves in relation to the liquid, then the liquid must also move in relation to a fixed solid, the fixed solid in this case being the walls of the cell tube. In order to compensate for this movement of liquid (known as electro endosmosis, and manifested as a flow along the walls of the tube, with return

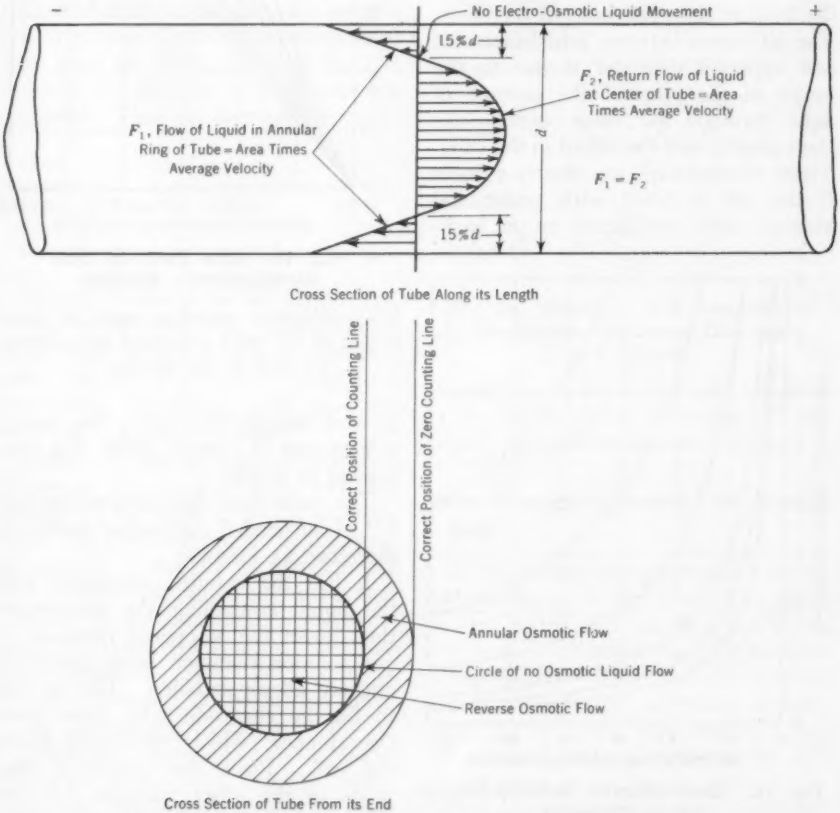
–
+
No Electro-Osmotic Liquid Movement
15%d
15%d
d
F_2, Return Flow of Liquid at Center of Tube = Area Times Average Velocity
F_1, Flow of Liquid in Annular Ring of Tube = Area Times Average Velocity
$F_1 = F_2$
Cross Section of Tube Along its Length
Correct Position of Counting Line
Correct Position of Zero Counting Line
Annular Osmotic Flow
Circle of no Osmotic Liquid Flow
Reverse Osmotic Flow
Cross Section of Tube From its End

Fig. 13. Electroosmotic Flow of Liquid in a Circular Tube

To avoid errors caused by electroosmotic flow of liquids, electrophoretic mobility must be measured at ring of no liquid flow.

at its center) it is necessary to measure the rate of travel of discrete particles at a distance from the tube wall of 15 per cent of the tube diameter. This relationship is shown in detail in Fig. 13. It should again be emphasized that whereas the particles to be counted must be at a distance from the wall representing 15 per cent of actual tube diameter, it is necessary first to focus the microscope at exact middepth of the cell, then to measure 15 per cent of the distance from the front wall of the tube of the *apparent* cell diameter. The difference between actual diameter and apparent diameter is due to the optics of the system (the passage of light through the three media—air, clear plastic, and the liquid in the cell). These relationships are clearly evident if the cell is filled with water and viewed with newsprint as a background beneath the cell. The letters will appear of normal width, but contracted in height.

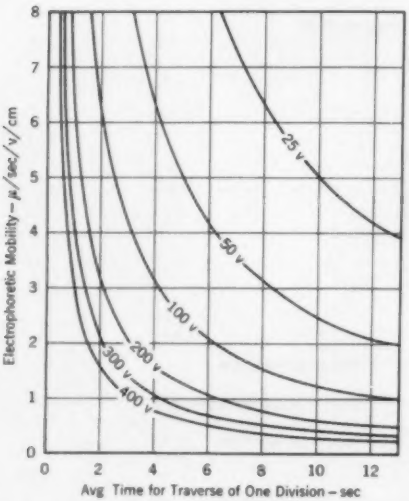

Fig. 14. Electrophoretic Mobility Curves for 6× Objective

Curves are for equipment with special ocular micrometer.

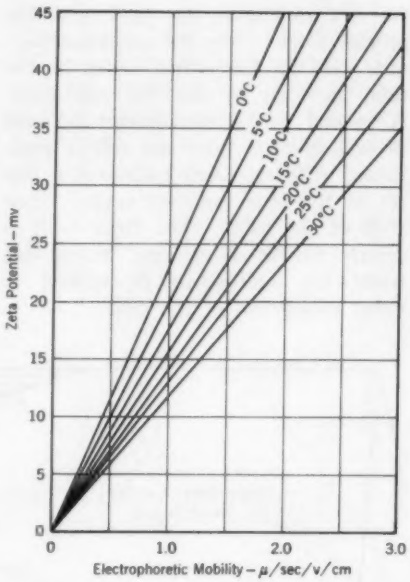

Fig. 15. Zeta Potential and Electrophoretic Mobility

Electrophoretic mobility may be converted to ZP with reference to temperature of the sample.

The procedure for determining the ZP of suspended particulate matter is therefore as follows:

1. Fill the solution chambers with the liquid containing the suspension, then insert the stoppered electrode in one solution chamber, being careful to exclude all air bubbles. Tilt the cell to fill the electrode tube, then insert the stopper, which effectively closes the cell. Place the open-type electrode in the other solution chamber.

2. Position the cell on the cell holder and adjust the mechanical stage so that the appropriate micrometer line of the

ocular micrometer coincides with the apparent front wall of the tube as shown in Fig. 12. Using the 6× eyepiece, focus the stereoscopic microscope to mid-depth of the cell. At this point the positioning line stands out sharply.

3. Connect the electrodes of the electrophoresis cell to the d-c power supply, and adjust—for example, to 300 v.

4. By means of the electrical timer, measure cumulatively the time (in seconds and tenths) required for each of ten or twenty particles (situated on or very near the third counting line) to traverse one division of the ocular micrometer. The basic data are therefore:

Voltage	300 v
Objective	6×
Number of particles counted	10
Total time	30.0 sec
Temperature of sample	15°C

Figure 14 shows curves for the 6× objective. Extend the 3.0 interval on the x axis (average time in seconds) upward to the 300 v curve. The electrophoretic mobility is then read directly as 1.40 μ/sec/v/cm.

Figure 15 makes possible the conversion of this electrophoretic mobility reading (1.40) directly to ZP in millivolts. As the temperature of the sample was 15°C, the ZP is therefore 22 mv—a reasonable value for raw water colloids. If these colloids migrated toward the positive electrode, they are electro-negative. The ZP value is normally rounded to the nearest unit.

The curves shown are typical. It is obviously necessary to prepare curves for the 4×, 6×, and 8× objectives, and log-log plotting produces straight lines rather than hyperbolas. The 2× objective is for calibration purposes only. The preferable objective for most work is 6×.

Both the cell and the technique are simple. The cell may be readily cleaned with any mild detergent, and the tube cleaned with a conventional pipe cleaner.

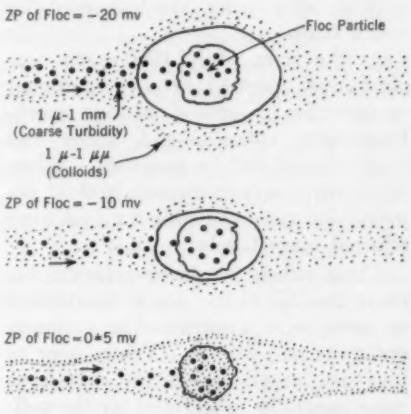

Fig. 16. Effect of Zeta Potential on Flocculation of Coarse Turbidity and Colloids

Coarse particles can become imbedded in floc despite ZP of more than ±5 mv, but colloids are repelled.

Zeta Potential Control at Waterford

Zeta potential measurements are run each morning at the Waterford plant of raw water colloids and floc from the first flocculation bay. Chemical dosage (alum-calcite and a cationic polyelectrolyte) are adjusted accordingly. This plant was designed for ZP control, and is operated by ZP control only.

Flocculation has remained excellent, even during the winter months, and the effluent is clear and sparkling.

The colloid concentration of the finished water is comparable to and sometimes less than distilled water. Taste and odor removal has been excellent, though the extremely cold winter of 1960–61 did produce some trying operating situations. At one time the flocculators were jammed by a 14-in. layer of ice which covered the entire basin.

At the start (July 1960), the new plant was operated in a conventional manner for a period of one week. Raw-water colloids—and tastes and odors—could not be removed. When ZP control was instituted, colloid removal and taste and odor removal were effected immediately.

These effects are being achieved because the ZP of the floc is maintained at zero, with a permissible variation not to exceed ±5 mv. When some of the best anionic coagulant aids were experimentally substituted for the cationic polyelectrolyte (which is added a few minutes after application of the alum-calcite mixture), floc formation remained excellent, but colloids were not removed, and neither were taste and odor. A high degree of colloid removal is completely synonomous with a high degree of taste and odor removal. It must be borne in mind that tastes and odors reflect molecular emissions from surfaces, and that surface areas of colloids are tremendous. A cube 1 mm on a side has a surface area of 6 sq mm, whereas if this cube is subdivided into 1-μ cubes (the largest size in the colloid range), this area becomes a thousand times larger, and increases a millionfold when the lower end of the colloid range is reached.

An incidental benefit that has been derived from ZP control is the removal of a considerably greater percentage of bacteria than can be effected with conventional coagulation. As water bacteria have dimensions approximating a few microns and ZP's approximating −15 to −35 mv, it is natural that maintaining ZP near zero results in their enmeshment in the floc to a much higher degree than is possible in normal coagulation. At Waterford approximately 99.9 per cent of the raw water coliform organisms (MPN is approximately 70,000) are enmeshed in the floc during coagulation, and are subsequently removed by sedimentation.

Hypothesis on Coarse Turbidity and Colloid Removal

It is necessary to form some hypothesis to account for the known condition that conventional coagulation at ZP's of −14 to −20 mv effectively removes the coarse fraction of turbidity (1 μ and larger), but not the fine fraction (1 μ and smaller); whereas with ZP controlled at near zero, both fractions are largely removed.

Figure 16 shows a concept of floc with ZP's adjusted to 20 mv, 10 mv, and 0 mv by appropriate additions of alum plus a cationic polyelectrolyte. The floc is veritably tumbling in a sea of colloids. At −20 mv, both the floc and the colloids are strongly negative, and few colloids can reach or become imbedded in the floc. At −10 mv, the mutually repelling forces are reduced, but the colloids, for the most part, still score a near miss. At zero ZP the colloids and floc not only do not repel one another, but as mechanical agitation brings them into physical contact, the London–Van der Waals forces come into effect, exerting mutual attraction.

The laws of force and motion seem applicable to this situation. Consider two particles of turbidity in a flocculation basin, impinging on a single particle of floc due to the mechanical mixing action created by the flocculator. Assume that all three particles are electronegative with ZP values of −15 mv; that the floc particle has a diameter of 1 mm; that the coarse turbidity particle has a size of 100 μ; and that the colloid has a diameter of 0.1 μ. The floc particle is therefore ten times the size of the coarse turbidity particle and 10,000 times the size of the colloid. The two turbidity particles therefore have a size ratio of 1,000:1. A sphere with a radius of 1 μ and sp gr of 2 weighs 8×10^{-12} g, whereas a similar sphere of 0.001 this size weighs 8×10^{-21}. The weight ratio is therefore one billion to one.

The kinetic energy of a particle of moving mass is equal to half the product of its mass and velocity squared, and the momentum is equal to the product of its mass and velocity. It is obvious, therefore, that as the two turbidity particles have approximately equal velocities (this being a function of the speed of mechanical agitation), the particle with the high mass is much more capable of driving through the repelling barrier created by the ZP of −15 mv or −10 mv, than the colloid of low mass. A mass difference of 1,000:1 would seem adequate to account for this condition, and the ratio of one billion to one (as with the two particles in question) seems more than ample. One can pitch buck shot into an electric fan, but not pith balls.

There are several highly pertinent subjects that have not been discussed in this paper for lack of space. These include the cationics—their nature, their possible dangers, their present availability,[9-11] and their effectiveness —and the cost aspects of ZP control. Cost has been and is receiving a considerable amount of attention. Early indications were that costs might be an almost prohibitive factor. It is now believed, however, that continuing development work augurs well for the economical use of the cationics.

It is not believed that highly polluted raw waters can be successfully treated (from the standpoint of effective colloid removal) by using only anionic or nonionic surface-active agents. After the Waterford plant had operated with complete satisfaction for a period of about one month, the cationic was intentionally shut off for several hours and an effective anionic was substituted. Although floc formation was excellent, colloid removal, and hence taste and odor removal, were so poor as to necessitate draining both the flocculation and sedimentation basins.

Many of the cationics on the present market (developed principally for the textile industry) leave much to be desired in terms of effectiveness in ZP control. Cost per pound and the percentage of active ingredients are both meaningless criteria. Evaluation can be made only on a basis of cost per unit change of ZP.

In regard to another subject on which the author has been queried many times—whether ZP determination augments the jar test, or vice versa—the answer is that the jar test augments ZP determination. The conditions of precipitation and dispersal can be indicated by ZP better than by pH.

It is hoped that this and other articles will elicit further interest in this

field. Continued research, particularly with waters containing highly electronegative colloids, as well as those containing colloids with a wide spread of ZP values, will undoubtedly show some variations in the pattern set forth. It is believed, however, that this pattern, established by research and fully confirmed by operation of the Waterford plant, is basic. Water coagulation is but one of the many processes affected by ZP.

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Modified Stannous Chloride Reagent for Orthophosphate Determination

Owen Sletten and Clayton M. Bach

A contribution to the Journal by Owen Sletten, Consultant, St. Louis, Mo., and Clayton M. Bach, Supt. of Water Treatment, Water Dept., Minneapolis, Minn.

AN inherent weakness in the stannous chloride method now being used for the determination of orthophosphate is the instability of the stannous chloride reagent. In order to prolong the shelf life of this solution, a number of preservative techniques have been suggested.[1, 2] These include storage of the reagent in a cool place, in a refrigerator, or in an aspirator bottle; floating the reagent on a layer of mineral oil; and the addition of mossy tin. The reagent may be modified to give excellent shelf life without the need for preservative techniques.

1. General Discussion

Experimental storage of stannous chloride solution for 5 years has demonstrated its excellent stability, even though no precautions were taken against its deterioration from light or heat. An investigation was also made to determine if 690 mμ was a satisfactory wavelength for measuring the color photometrically, as set forth in *Standard Methods*.[2] The results indicate that the prescribed wavelength is satisfactory with the use of the modified reagent (Fig. 1). The proposed modification is designed to follow the procedure as set forth in *Standard Methods*,[2] with only slight procedural differences.

2. Reagents

Of the reagents prescribed, only the stannous chloride solution is to be modified.

Stannous chloride solution. Dissolve 2.5 g of a *fresh* supply of $SnCl_2 \cdot 2H_2O$ in 100 ml of reagent grade glycerine. Heating in a water bath and stirring with a glass rod are recommended to hasten dissolving. This reagent is stable, requiring neither the addition of preservatives nor special storage.

3. Procedure

3.1. If necessary, filter the sample. To a 50-ml sample that contains no

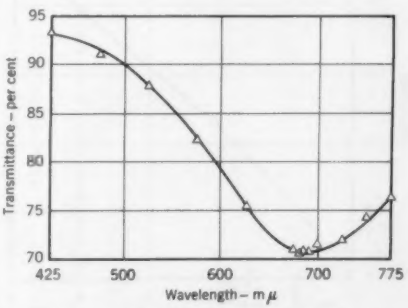

Fig. 1. Determination of Optimal Wavelength

The results showed that a wavelength of 690 mμ is satisfactory for measuring the color photometrically.

more than 0.30 mg PO_4 and no color and turbidity, add 1 drop of phenolphthalein indicator. If the sample turns pink, add a solution of strong acid, drop by drop, until the color is removed.

3.2. Add, with thorough mixing after *each* addition, 2.0 ml ammonium molybdate solution (I) and 0.25 ml (4 drops) stannous chloride solution. The rate of development of color and its intensity depend on the temperature of the final solution. Each increase of 1°C produces about a 1 per cent increase in color. For that reason, samples, standards, and reagents should be within 2°C of one another and at a temperature of 25°C±5°.

3.3. After 12 min ±2 min, using the same time interval between determinations as before, measure the color photometrically at a wavelength of 690 mμ, and compare the measurement with the calibration curve, using a distilled-water blank to which have been added the reagents. Alternatively, use 50-ml nessler tubes and make a visual comparison against standards prepared simultaneously.

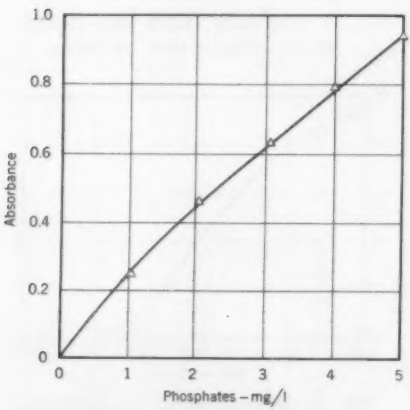

Fig. 2. Absorbance of Phosphates

The curve shown is a plot of the data in Table 1.

TABLE 1

Absorbance Values for Phosphate

PO_4 Concentration *mg/l*	Samples							Mean
	A	B	C	D	E	F	G	
	Absorbance—*1,000's*							
1	230	240	255	205	240	220	250	235
2	460	450	445	440	430	440	450	445
3	650	620	640	660	590	630	600	625
4	780	780	800	800	780	785	780	790
5	950	950	920	950	920	970	930	945

3.4. A blank must always be run on the reagents and distilled water. Inasmuch as the color at first develops progressively and later fades, it is essential that the timing be the same for samples as for standards. The blank, which consists of distilled water to which are added the reagents, should be free from color. Color in the blank may be the result of improperly washed glassware, impurities in the ammonium molybdate, or prolonged storage of the molybdate solution. It should be noted that the calibration curve may deviate slightly from a straight line, even with concentrations less than 6 mg/l.

The data in Table 1 are plotted in Fig. 2 and were obtained in phosphate determinations, with standards in the range of 0–5 mg/l as PO_4, with the use of a spectrophotometer * having a sensitivity number of 3, a wavelength of 690 mμ, a slit opening of 0.3 mm, and a lightpath of 1 cm.

* The spectrophotometer used was a Beckman Model B, made by Beckman Instruments, Fullerton, Calif.

Samples A, B, D, and E in Table 1 were run with freshly made ammonium molybdate solutions and freshly made nonglycerated stannous chloride reagents. Samples C, F, and G were run with freshly made ammonium molybdate solutions and glycerated stannous chloride reagents prepared in 1956.

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CHAPTER 7

Water Hammer and Surge

AWWA Steel Pipe Committee Report

A report on design and installation of steel water pipe has been prepared by AWWA Committee 8310 D—Steel Pipe. The report is being published in installments (May 1961 issue, p. 632; June 1961 issue, p. 750), with the expectation that it will be reprinted as a unit when complete. Comment and discussion by readers are encouraged. Discussion should be addressed to the Editor of the JOURNAL.

THE effect of the propagation of pressure waves in closed conduits containing water has come to be known by the rather homely name of "water hammer." If a self-closing valve at the wash bowl or the kitchen sink becomes loose and snaps shut, a chug is heard and felt, as if someone had hit the pipe with a hammer. This chug doubtless is responsible for the term "water hammer," which has become a generic one, meaning not only the effect but also the entire phenomenon, of wave propagation in closed conduits.

The phenomenon of water hammer and its practical significance to the operators of water works have not been very widely understood. This condition has prevailed in spite of the many contributions to the literature that have been made on this subject by members of the engineering profession. Theories and formulas developed for computing the effects of sudden changes of velocity in fluid-conducting pipelines are more readily applied to design than to field operations, because of lack of data on the characteristics of the valves ordinarily used to control flow.

The control of water hammer is a subject involving much speculation and widely differing opinions. Many engineers and operating men, as well as a number of manufacturers, associate water hammer with the mechanical slamming of a swing check valve, believing that if there is no noise, there is no water hammer. This belief, which is far from the truth, has lulled many engineers into thinking that their water hammer problems were nonexistent, only to find later that a line had pulled apart at the joints or even split wide open.

To control water hammer one must first understand what it is and what causes it and then determine the magnitude of the surge and its relationship to the strength of the pipe at all critical points. Much mystery has surrounded the solution of water hammer problems, probably because of the complex theory of wave motion involved and the lack of time or opportunity for most engineers to study the details or to perform the

calculations. As a result, a number of approximate formulas have been developed, some of which are good within limits, but most of which are dangerous to use. The variations from the true values in a given case range from 35 per cent below to more than 500 per cent above. For a given set of conditions, Table 7.1 compares a number of such approximate formulas found in textbooks and handbooks, with the well proved elastic-wave theory of Joukovsky (*16*) and Allievi (*3*).

Such variations led to a study of water hammer by the American Society of Mechanical Engineers, the American Water Works Assn., and

TABLE 7.1

Comparison of a Number of Approximate Formulas for the Determination of Pipeline Surges

EXAMPLE: length (L), 820 ft; velocity (V), 11.75 fps; time (T), 2.1 sec; head (H_0), 165 ft; wave velocity (a), 3,220 fps; critical time $\left(\frac{2L}{a}\right)$, 0.509 sec.

Name of Formula	Formula	Max. Pressure Rise, *ft*	Percentage of Max. by Elastic Wave Theory, *per cent*
Vensano	$h = \frac{2LV}{gT}$	285.0	129.6
Warren	$h = \frac{LV}{g\left(T - \frac{L}{a}\right)}$	162.3	73.8
Johnson	$h = \frac{LV_0}{2g^2H_0T^2}\left[LV_0 + \sqrt{4g^2H^2T^2 + L^2V_0^2}\right]$	216.0	98.3
Fanning	$h = \frac{LV}{gT}$	142.5	64.8
Talbot	$h = 2.31V\sqrt{\frac{1}{1 + \frac{Kd}{Ee}}}\sqrt{\frac{WK}{144g}}$	1,175.0	535.0
reduces to:	$h = \frac{aV}{g}$		
de Sparre	$h = \frac{2LV_0}{gT}\frac{1}{2\left[1 - \frac{LV_0}{2gTH_0}\right]}$	251.0	114.2
Elastic Wave Theory:			
a. Detailed calculation—equations of Allievi, Gibson, Quick and Gibson arithmetic integration		219.89	100.0
b. Charts by Quick and Allievi		219.9	100.0

From Kerr (17)

other societies beginning in 1931. Two symposiums were published, one in 1933 and one in 1937 (*1*, *2*). Through these studies, and through actual tests of installations, the elastic-wave theory has been completely proved.

7.1. Basic Relationships

It may be well at this point to state a few of the fundamental relationships in surge wave theory which determine the magnitude of the pressure rise and its distribution along a conduit. These comments relate to gravity flow or discharge lines from centrifugal pumps only:

TABLE 7.2

Velocity of Pressure Wave for Large-Diameter Steel Pipe

Diameter / Thickness, $\frac{d}{e}$	Wave Velocity, a, fps
100	3,310
120	3,150
140	3,020
160	2,900
180	2,800
200	2,700
250	2,470

a. The pressure rise for instantaneous closure is directly proportional to the fluid-velocity (V) cutoff and to the magnitude of the surge wave velocity (designated a) and is independent of the length of the conduit. Its value is:

$$h = \frac{aV}{g} \text{ (ft of water)} \quad \ldots\ldots (7.1)$$

or

$$p = \frac{aWV}{144g} = \left(\frac{a}{g}\right)\left(\frac{\text{sp gr}}{2.3}\right) V \text{ (psi)} \quad \ldots (7.2)$$

in which

$$a = \text{wave velocity} = \frac{12}{\sqrt{\frac{W}{g}\left(\frac{1}{k} + \frac{d}{Ee}\right)}} \text{ (fps)} \quad \ldots . (7.3)$$

In the above equations:

a = wave velocity (fps)
h = pressure rise above normal (ft of water)
p = pressure rise above normal (psi)
V = velocity of flow (fps)
W = weight of fluid (lb/cu ft)
sp gr = specific gravity of fluid (water = 1.0)
k = bulk modulus of compressibility of liquid (psi)
E = Young's modulus of elasticity for pipe wall material (psi)
d = inside diameter of conduit (in.)
e = thickness of conduit wall (in.)
g = acceleration due to gravity (32.2 fps/sec).

Also:

L = length of conduit (ft)
$\frac{2L}{a}$ = critical time of conduit (sec)
T = closing time (sec).

For steel pipe, Eq 7.3 reduces to:

$$a = \frac{4{,}660}{\sqrt{1 + \frac{1}{100}\cdot\frac{d}{e}}} \text{ (fps)} \quad \ldots\ldots (7.4)$$

using $k = 294{,}000$ psi and $E = 29{,}400{,}000$ psi.

Figure 7.1 gives values of pressure wave velocity for various pipe materials with $\frac{d}{e}$ ratios up to 100.

For steel pipe, higher ratios are frequently encountered in large sizes, and Table 7.2 gives computed values up to $\frac{d}{e} = 250$.

b. When the flow rate is changed in a time greater than zero but less than or equal to $\frac{2L}{a}$ sec, the magnitude of the pressure rise is the same as with instantaneous closure, but the duration of the maximum value decreases as the time of closure approaches $\frac{2L}{a}$ sec.

c. Under the conditions in Paragraph *b*, the pressure distribution along the pipeline varies as the time of closure varies. The pressure decreases uniformly along the line if closure is in $\frac{2L}{a}$ sec. The maximum pressure at the control valve exists along the full length of the line with instantaneous closure, and for slower rates travels up the pipe a distance equal to $L - \frac{Ta}{2}$ ft, then decreases uniformly.

d. The surge pressure distribution along the conduit is independent of the profile or ground contour of the line so long as the total pressure remains above the vapor pressure of the fluid.

e. For closing times greater than $\frac{2L}{a}$ sec, the maximum pressure rise will be a function of the maximum rate of change in flow with respect to time, $\frac{dV}{dT}$.

f. Nonlinear closures can be investigated, and the proper valve timing to hold the maximum pressure rise to any desired limiting value can be determined.

g. The effect of centrifugal pumps and quick-closing check valves or control valves can be investigated using the graphical method of analysis.

h. The profile of the conduit leading away from a pumping station may have a major influence upon the surge conditions. The minimum effect will be found when the entire line is level, the maximum when the line is steep or when high points occur along the line.

i. Parting and rejoining of the liquid column can produce extremely high pressures and may cause failure of the conduit (*20*).

j. The effect of friction can be accounted for graphically in any surge problem and, when such losses are less than 5 per cent of the normal static or working pressure, usually can be neglected.

k. The greater the degree of accuracy desired for the results of a surge analysis, the more must be known about the various hydraulic and physical characteristics of the system.

l. The agreement between theoretical analyses and actual field test results has confirmed the accuracy of the elastic-wave theory in water conduits.

The velocity of the pressure wave, a, is a fundamental factor in any surge study, as the surge pressures are directly proportional to its value.

This velocity is dependent upon the pipe diameter, wall thickness, material of the pipe walls, and density and compressibility of the fluid in the pipe.

Knowledge concerning the physical characteristics of the pipe material is fairly complete. Young's modulus for steel lines can be taken at 30,000,000 psi as an average between 29,000,000 and 31,000,000 psi. If the ratio of diameter to thickness is known, it is necessary to know only the density and the compressibility of the liquid within the pipe to determine the surge wave velocity, *a*.

Within the range of ordinary operating temperatures for water, 32°–100°F, and for pressures from 0 to 1,000 psi, the specific gravity can be taken at 1.00. In the same range, the modulus of compressibility, or "bulk modulus," has been found from measurement checked by field tests to be approximately 300,000 psi with a variation of ±3 per cent (*15*).

7.2. Checklist for Pumping Mains

A few factors can be checked to indicate whether surges of serious proportions will occur in any given system, once the physical, hydraulic, and operating characteristics are established. For most transmission mains supplied by motor-driven centrifugal pumps, the following twelve questions will give a clue to the seriousness of the surge problem (*13, 17*):

a. Are there any high spots on the profile of the transmission main, where the occurrence of a vacuum can cause a parting of the water column when a pump is cut off?

b. Is the length of the transmission main less than 20 times the head on the pumps (both values expressed in feet)?

c. Is the maximum velocity of flow in the transmission main in excess of 4.0 fps?

d. Is the factor of safety of the pipe less than 3.5 for normal operating pressures?

e. What is the natural rate of slowing down of the water column if the pump is cut off? (Will the column come to rest and reverse its direction of flow in less than the critical surge wave time for the transmission main?)

f. Will the check valve close in less than the critical time for the transmission main?

g. Are there any quick-closing automatic valves set to open or close in less than 5.0 sec?

h. Would the pump or its driving motor be damaged if allowed to run backward up to full speed?

i. Will the pump be tripped off before the discharge valve is fully closed?

j. Will the pump be started with the discharge gate valve open?

k. Are there booster stations on the system which are dependent on the operation of the main pumping station under consideration?

l. Are there any quick-closing automatic valves used in the pumping system that become inoperative with the failure of pumping system pressure?

If the answer to any one of Questions *a*–*g* is affirmative, there is a strong possibility that serious surges will occur. If the answer to two or more of the twelve questions is affirmative, surges will probably be experienced and their severity will be in proportion to the number of "yes" answers.

7.3. General Studies for Water Hammer Control

One of the two questions which are usually asked about advance studies on surges is: How can such studies be undertaken during the design stage? The answer is simple. Once the general layout of the system has been made, the length, diameter, thickness, material, and capacity of the pipe, as well as the type and size of pumps, can be established. The normal operating pressures at various points in the system can be computed and the allowable maximum pressures fixed. By this means, the margin for water hammer can be found. The design should then be adjusted to provide either safety factors large enough to withstand such conditions as might be encountered or suitable remedial or control devices. The latter method is usually the less costly. It is important to note that there is no single magic device that will cure all surge difficulties. Only by a study of both normal operating conditions and possible emergency conditions can the proper valve-timing, surge-damping, or relief devices be selected.

It is not feasible to make general recommendations on the type, size, and application of surge control equipment for all plants. Several possible solutions should be considered for any individual installation, and that one selected which gives the maximum protection for the least expenditure. Surges can often be reduced substantially by using bypasses around check valves, by cushioning check valves for the last 15–20 per cent of the stroke, or by adopting a two-speed rate of valve stroke. Water hammer resulting from power failure to centifugal pumps can sometimes be held to safe limits by allowing the pumps to run backward. Air inlet valves may be needed, or the preferred solution may be to use a surge tank, a surge damper, or an air chamber. In a number of plants, no devices will be required to hold the pressure rise within safe limits.

It is essential to coordinate all the elements of a system properly and to ascertain that operating practices conform to the requirements for safety. As changes take place in the system demand, it may be necessary to review and revise the surge conditions, particularly if the capacity is increased, additional pumpage or storage is added, or booster stations are planned.

TABLE 7.3

Empirical Allowances for Water Hammer in Cast-Iron Pipe

Diam. of Pipe *in.*	Water Hammer *psi*	Diam. of Pipe *in.*	Water Hammer *psi*
4–10	120	24	85
12–14	110	30	80
16–18	100	36	75
20	90	42–60	70

From Barnard (23)

The second common question on advance surge studies is: How effective are the recommended solutions to the problem when the equipment is placed in operation? If a competent investigation was made during the design stage and the recommendations arising from it have been carried out, the final plant is almost always operated without damage due to water hammer. The agreement between the theoretical analyses, properly applied, and the actual tests of installations has been extremely close. When a surge study was not undertaken and dangerous conditions existed, there have almost invariably

been serious surges, and sometimes costly damage has resulted. The time and effort spent on a surge study in advance of the final design is the least expensive means of insuring against surges. The elastic-wave theory has been completely proved in actual practice, and it remains only for design engineers to take the initiative in making these studies and installing surge control devices without waiting for serious failures to occur.

7.4. Standard Allowances for Water Hammer

Divergent opinions (*21*) are held on the merits of using for all types of pipe the allowances for water hammer that have been used for cast-iron pipe. The current standard allowances for cast-iron pipe seem to stem from work done in the early 1890's and are generally attributed to Dexter Brackett. The recommended values for cast-iron pipe vary from 120 psi for 4-in. pipe to 70 psi for 60-in. pipe (see Table 7.3). There is no adjustment in the allowances for variations in thickness for any given diameter, but merely a flat allowance above the normal working pressures.

Many conditions have changed since the empirical allowances originated. Automatic stop, check, and throttling valves were not then as widely used as they are today. Valve closures measured in seconds and motor-driven centrifugal pumps were

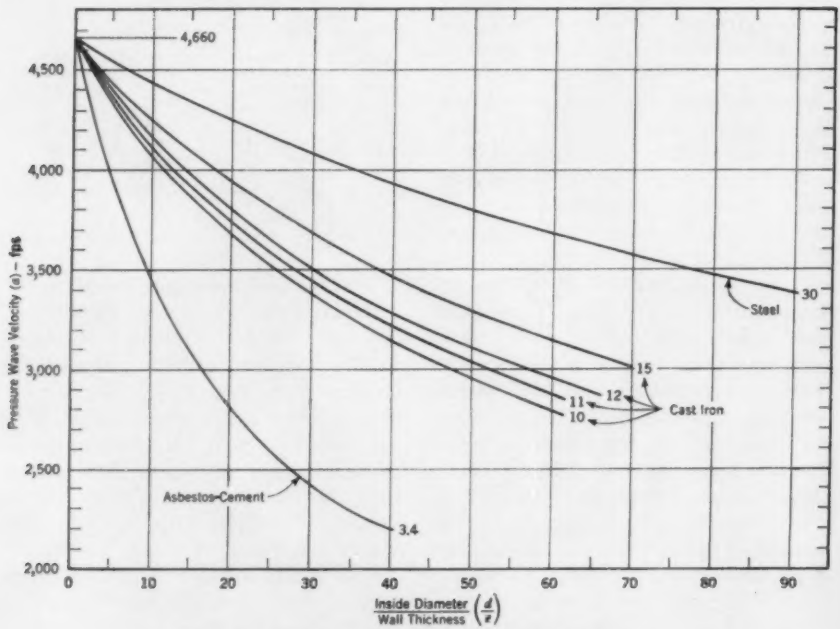

Fig. 7.1. Surge Wave Velocity Chart for Water

The numbers at the right of the curves represent the modulus of elasticity (E) *in 1,000,000-psi units for various pipe materials* (17).

practically unknown. New types of pipe have since been introduced and used. Consequently, it is questionable whether standard allowances for water hammer should be applied universally to all types of installations. Nor can it be said that such allowances will provide full security under all circumstances. Water hammer can and should be considered in the

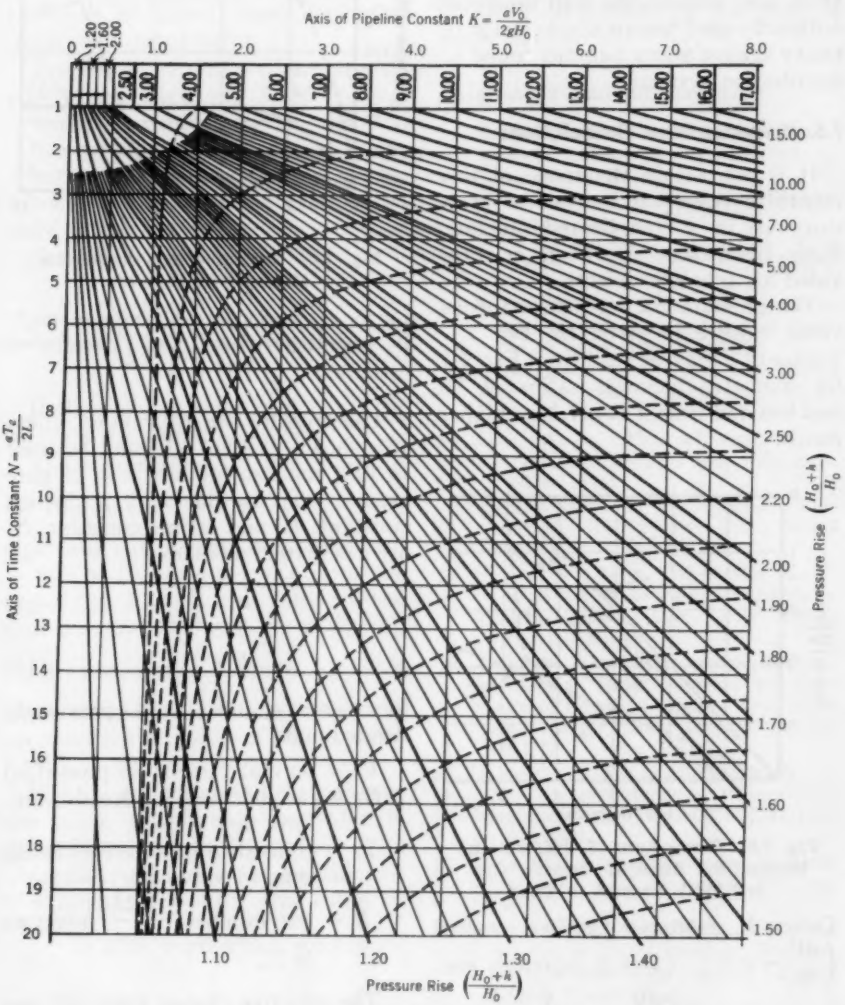

Fig. 7.2. Maximum Pressure Rise for Uniform Conduits

The diagram may be used to find the rise for any rate of gate closure (3). *See text* (Sec. 7.5) *for explanation of letter symbols.*

design of pumping station piping, force mains, and long supply lines. Suitable means should be provided to reduce its effect to the minimum that is practicable or economical.

The consensus is that, for steel pipe 18 in. and smaller, the wall thickness ordinarily used has an ample factor of safety against water hammer found in distribution systems.

7.5. Pressure Rise Calculations

It is not within the scope of this report to cover an analysis of pressure rise in a complicated pipeline. Some basic data are, however, provided for simple problems.

The pressure rise for instantaneous valve closure is given by Eq. 7.1. Values of a may be read from Fig. 7.1 for diameter-thickness ratios of 90 and less and from Table 7.2 for higher ratios.

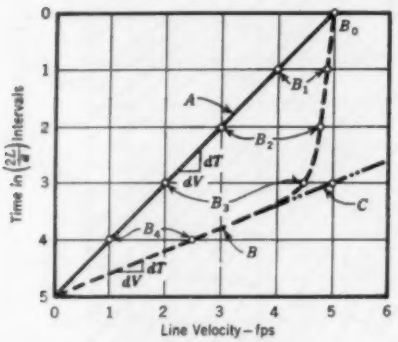

Fig. 7.3. Comparison of Uniform and Nonuniform Rates of Velocity Control With Respect to Time

Curve A *represents uniform control* $\left(\frac{dV}{dt} = 0.50\right)$; *Curve* B *represents nonuniform control* $\left(\frac{dV}{dt} = 1.25\right)$; *and Curve* C *is based on uniform equivalent time.* B_0–B_4 *are points of critical time* (17).

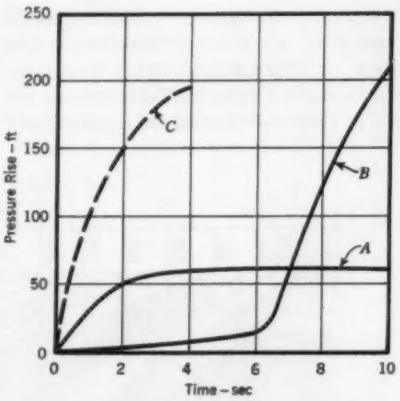

Fig. 7.4. Pressure-Time Curves for Example in Fig. 7.3

Curves are shown for: A, *uniform cutoff;* B, *nonuniform cutoff; and* C, *equivalent time* (17).

The pressure rise diagram of Allievi (*3*), shown in Fig. 7.2, may be used to find the rise for any rate of gate closure. To use the chart, a pipeline constant, K, and a time constant, N, are calculated, as follows:

$$K = \frac{aV_0}{2gH_0} \quad \ldots\ldots\ldots\ldots (7.5)$$

$$N = \frac{aT_e}{2L} \quad \ldots\ldots\ldots\ldots (7.6)$$

in which a, g, and L are as previously defined and:

V_0 = normal velocity in pipe (fps)
H_0 = normal head for steady, uniform flow (ft)
T_e = equivalent or effective closing time of valve (sec)
N = the number of $\frac{2L}{a}$ intervals in T_e.

The effective closing time (*17*; pp. 989–91) of a valve is the time it would take to cut off flow if the maximum rate at any portion of

the stroke were applied uniformly throughout the stroke. Figure 7.3 shows this relationship and Fig. 7.4 shows the pressure-time curves for uniform and nonuniform rates of flow control. T_c may vary from 35 to 55 per cent of travel time, depending upon the type of valve.

Problem 1. A 36-in. diameter steel pipe, $\frac{1}{4}$ in. thick and 5,280 ft long, is under a normal head of 100 ft and is flowing full with a velocity of 10 fps. What is the pressure rise for instantaneous valve closure at the outlet end?

Solution. From Eq 7.3 and 7.1:

$$a = 3{,}000 \text{ fps}$$

$$h = \frac{3{,}000 \times 10}{32.2} = 930 \text{ ft (or 403 psi)}$$

If the velocity were only 1 fps, the pressure rise would be 40 psi.

The critical time, or time constant, for this conduit, $\frac{2L}{a}$, is $3\frac{1}{2}$ sec. The pipe constant, K, equals $(3{,}000 \times 10) \div 64.4 \times 100$, or 4.65. Entering the top of Fig. 7.2 with this value for one time constant (top grid line in figure), the pressure rise $\frac{H_0+h}{H_0}$ is 10.2. Since $H_0 = 100$, $h = 920$ ft. This compares with the value of 930 ft from Eq 7.1.

Problem 2. What will the pressure rise be in this line if the valve is closed in six periods (21 sec)?

Solution. At the intersection of $N = 6$ and $K = 4.65$, the value of $\frac{H_0+h}{H_0}$ is 2.10, from which $h = 110$ ft.

7.6. Good Practice

The possibility of water hammer and surge should never be overlooked in design. Consider the answers to the list of questions in Sec. 7.2 and be guided accordingly for pumping mains. If in doubt, consult references or authorities. Refer to Sec. 6.7 (Chap. 6) for design criteria of stresses and steel to provide for the effect of water hammer.

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CHAPTER 8

Earth Loads on Steel Pipe

STEEL pipe, both with thin wall and with thick wall, has been used underground for nearly a century in the case of smooth-wall pipe and half a century in the case of corrugated pipe. These kinds of pipe have been subjected to earth loads and live loads of many kinds and of high intensity without any general failure by collapse or undesirable change in shape. Therefore, the load resistance of pipe in the commonly used diameters and wall thicknesses has been generally adequate and the problem of analysis is not a vital one, except that deflection of steel water pipe must be restricted to permissible limits for the linings and coatings used and for water tightness of the joint where mechanical couplings are used.

Real collapse failure of steel pipe does not occur under earth loads until a condition is reached where the vertical diameter has been decreased about 20 per cent of the nominal diameter and the horizontal diameter has been increased a similar amount. Many thousands of corrugated steel drainage structures show that they are virtually uncrushable under any existing fill or moving-wheel load with or without impact when the earth cover is at least equal in depth to the pipe diameter. The average vertical deflection of such structures, even including those under railroads, is about 2 or 3 per cent of the nominal diameter.

The Barnard "sidewise" settlement theory, described in sections beginning with 8.9, was derived from observations on steel water pipe. The theory is basic, however, and has been found to apply equally well to corrugated culvert pipe. Therefore, examples of both types are relevant to show validity through installation experience. All observations plotted in Fig. 8.6 pertain to steel water pipe installations, most of the lines being Dresser coupled. Vertical deflection averages about 2 per cent, which is normal for such lines, and is confirmed by the sidewise settlement theory.

8.1. Types of Conduits

When grouped according to degree of rigidity, three classes of pipe are generally recognized:

a. Rigid conduits, whose cross-sectional shapes cannot be distorted sufficiently to change their vertical or horizontal dimensions more than 0.1 per cent without causing material damage.

b. Semirigid conduits, whose cross-sectional shapes can be distorted sufficiently to change their vertical or horizontal dimensions more than 0.1 per cent but not more than 3.0 per cent, without causing material damage.

c. Flexible conduits, whose cross-sectional shapes can be distorted sufficiently to change their vertical or horizontal dimensions more than 3.0 per cent before causing material damage.

Steel pipe, having either light or heavy wall, because of its physical

characteristics can always function as a flexible conduit, with due consideration given for the relative rigidity of linings or coatings of water pipe.

The working data in this report have special application to conduits made of steel and not to conduits made of other materials.

8.2. Types of Loads

Earth loads on pipe are of two kinds. The one more common in water works practice is trench loading, the pipe being laid in an excavated trench and backfilled. The second kind occurs when a pipe is laid on a graded or prepared ground surface and a fill of earth is placed around and over the pipe. As the load carried by the pipe may differ in the two cases, trench loading should be distinguished from fill loading. Further, the reaction of flexible pipe to either fill or trench load is different from the reaction of rigid pipe to these loads.

8.3. Rigid Versus Flexible Pipe

The differences in reaction to load exhibited by rigid and by flexible or semiflexible pipe affect design. The methods of design which apply to one do not apply to the other.

Field load tests made on buried pipe show: [1] that the load developed on rigid pipe is greater than that developed on flexible pipe; and [2] that this greater load tends to concentrate at the top and bottom of the rigid pipe, thus creating bending moments which must be resisted by the shell. The tests show that the lesser load developed on flexible pipe: [1] is more evenly distributed around its circumference, with less concentration at any point than is true for rigid pipe; and [2] may be carried as a compressive force in the shell which, depending on stiffness, frequently is resisted more by arch action than by moment strength. The effect of earth

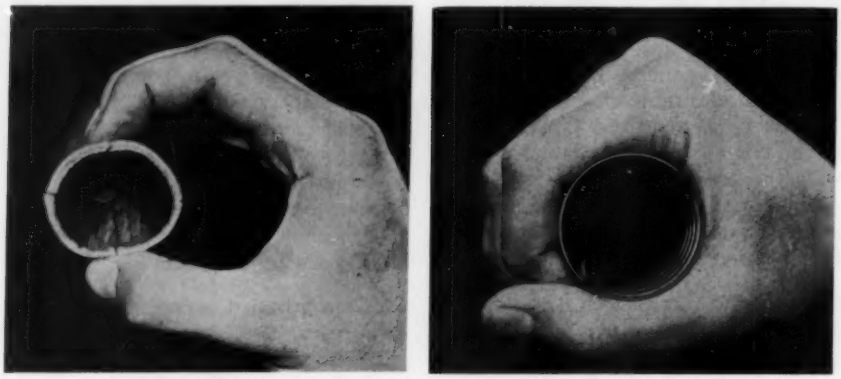

Fig. 8.1. Effect of Earth Load on Rigid and Flexible Pipe

The load on rigid pipe (demonstrated at the left) is concentrated at the top and bottom of the pipe, creating bending moments that must be resisted by the shell. With flexible pipe (demonstrated at the right), a lesser load is developed, because it is more evenly distributed around the circumference.

load on rigid and flexible pipe is shown in Fig. 8.1.

Although the maximum load-carrying capacity of flexible pipe depends to some extent on the wall thickness and its section modulus, the pipe, by deflecting, is able to make full use of the load-carrying ability of the earth surrounding it. As the pipe may change shape *without failure*, it transfers part of the vertical load into a horizontal or radial thrust which is resisted by the passive pressure of the earth at its sides as these move outward. If the wall itself were rigid, this movement could not occur without wall fracture. It follows that the rigid pipe must carry the whole load itself or break, whereas the flexible pipe divides the load with the earth enclosing it. Therein lies the inherent difference between rigid and flexible behavior and the explanation of why the classical bending-moment formulas apply to the analysis of rigid pipe but not to the analysis of flexible pipe.

Figure 8.2 shows photographs of sand models made to demonstrate what takes place when normal settlement occurs in a fill containing pipe. In these models, the horizontal lines were straight before settlement occurred. Their displacement gives a clue to what causes the difference in loading on rigid and flexible pipe.

In the left-hand portion of Fig. 8.2, showing rigid pipe, *S* represents the amount of settlement of fill by consolidation. The layers above the rigid pipe are higher than those at the sides. The outside layers, by sinking farther than the layers in the prism above the pipe, create a drag at the sides of the prism above the pipe and add to the prism weight the amount of the frictional forces in the two vertical friction planes. The load on the rigid pipe may therefore be greater than the weight of the earth prism directly above it.

In the right-hand portion of Fig. 8.2, showing flexible pipe, *D* represents the deflection of the pipe as it yields

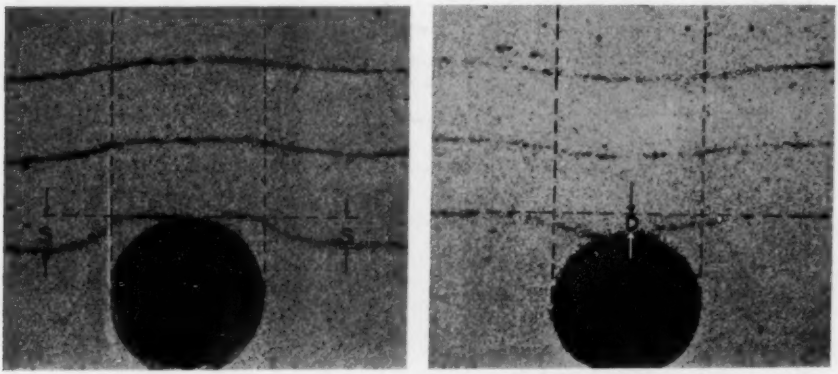

Fig. 8.2. Effect of Settlement on Rigid and Flexible Pipe in Fill

In the photograph at the left, pertaining to rigid pipe, S *represents the amount of settlement of fill by consolidation. In the photograph at the right, pertaining to flexible pipe,* D *represents the deflection of the pipe as it yields under load.*

under load. In this case, after settlement and consolidation of the fill, the layers above the pipe are *lower* than those at the sides. The weight of the earth prism resting on the flexible pipe is partially supported by the material at the sides of the pipe. This partial support is measured by the amount of the upward acting frictional force generated in the two friction planes and is governed, too, by the relative settlement of fill and deflection of pipe.

Consequently, in design, it should be considered that a rigid pipe under a fill or embankment may carry a load greater than the prism weight directly over it by the amount of frictional force drag between prism and side earth; a flexible pipe may carry a load which is less than the same prism weight by an uplift amount dependent on a similar frictional force. With rigid pipe the frictional force is additive. With flexible pipe it is subtractive. The same principle applies

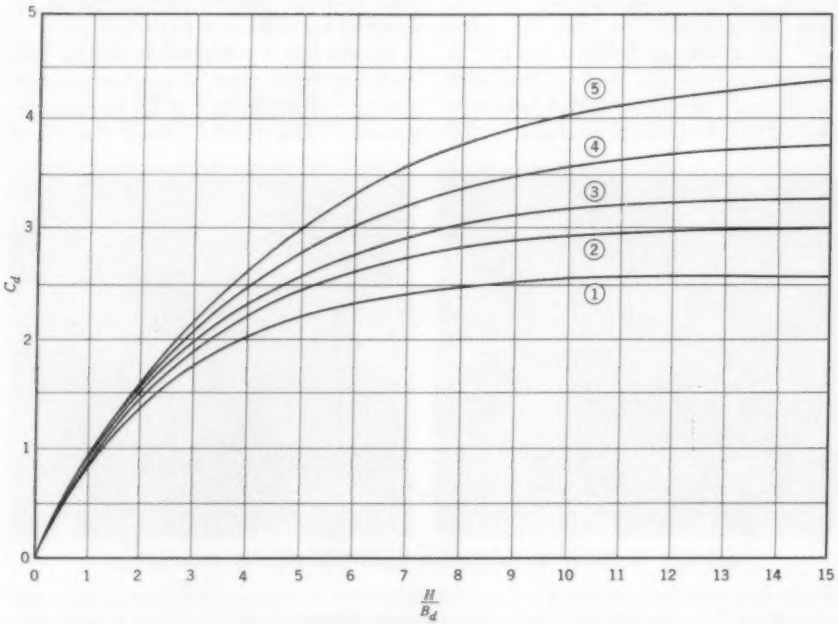

Fig. 8.3. Computation Diagram for Loads on Pipe Completely Buried in Ditches

C_d refers to the calculation coefficient; H, *to the height (ft) of fill over the top of the conduit; and* B_d *to the width (ft) of the ditch at the top of the pipe. The numbers on the curves represent: [1] minimum for granular materials without cohesion; [2] maximum for sand and gravel; [3] maximum for saturated topsoil; [4] ordinary maximum for clay; and [5] maximum for saturated clay.*

to flexible pipe in a trench, as explained below.

8.4. Determination of Load

The vertical load acting on a pipe under a fill or embankment may be calculated in accordance with Marston's formula applying to fill or embankment conditions (*1*). Alternatively, for flexible pipe, fill load may be taken as the weight of the earth prism above the pipe. The Marston formula for embankment or fill will not be discussed here.

When installation is made in a trench, the load may be calculated in accordance with Marston's formula for "ditch" conduits (*1*). Or, for flexible pipe, the load again may be taken as the weight of the earth prism above the pipe.

The Marston ditch conduit formula is:

$$W_c = C_d w B_d^2 \quad \ldots\ldots\ldots (8.1)$$

in which:

W_c = load on the conduit (lb/lin ft of pipe)
C_d = a calculation coefficient (see Fig. 8.3)
w = unit weight of ditch fill material (lb/cu ft)
B_d = width of ditch at top of pipe (ft).

This formula gives the total vertical load in the ditch or trench at the level of the top of the pipe. Marston and others have shown by experiment and observation of actual structures that practically all of this load is carried by the conduit itself when the conduit is rigid, as this type is very unyielding in relation to the earth materials between the sides of the structure and the sides of the trench.

If, however, the conduit is relatively flexible, such as a thin-walled steel pipe, it will deflect vertically and, by so doing, transmit a portion of the load to the trench walls by friction and to the trench bottom by settlement. The right-hand photograph in Fig. 8.2 illustrates the action of friction in reducing trench load if the trench sides are considered to be just outside the vertical lines. In Fig.

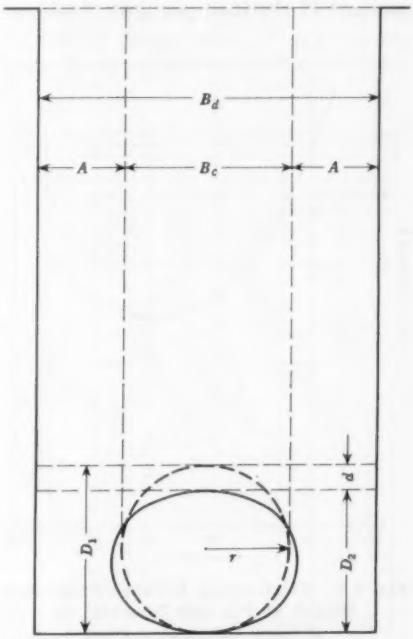

Fig. 8.4. Deflection of Steel Pipe

Pipe deflection transfers a portion of earth load to the sides and bottom of the trench. B_d *is the trench width at the top of the pipe;* B_c *and* D_1, *diameter of installed pipe;* A, *the difference between* B_d *and* B_c; d, *pipe deflection;* D_2, *diameter of deflected pipe; and* r, *radius of installed pipe.*

8.4, the trench fill is shown divided into three prisms. It is suggested that, as a *minimum* loading condition, if the pipe deflection d equals the settlement of side fill below the top of the pipe, the pipe and the side fills may each carry the same amount of the load per unit of width.

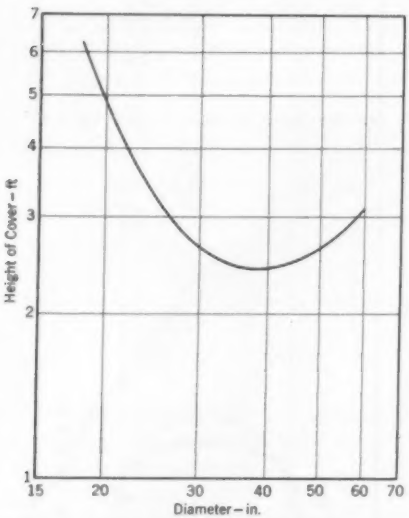

Fig. 8.5. Relationship Between Calculated Height of Fill and Diameter of 16-Gage Corrugated Pipe

The relationship was computed by use of Eq. 8.3 for a deflection of 1 per cent of pipe diameter; a constant e = *20; lag factor* = *1.5;* K = *0.10; and soil weight* = *120 lb/cu ft* (5).

Under these conditions, the load on the flexible pipe may be determined by multiplying Marston's load expression (Eq 8.1) by the ratio of the width of the pipe to the width of the trench, giving:

$$W_c = C_d w B_d^2 \left(\frac{B_c}{B_d}\right) \ldots\ldots (8.2)$$

in which B_c is the width of the pipe and the other symbols are as indicated for Eq 8.1.

It should be pointed out that, for steel pipe, Eq 8.1 represents the maximum loading conditions and Eq 8.2 represents the probable minimum loading condition which may develop on a pipe in a trench. The actual load in a given case may lie somewhere between these limits, depending upon the soil properties and the relative rigidity of the conduit compared with the side fills. For more rigid pipe, the actual load will approach that given by Eq 8.1, while, for quite flexible pipe, the actual load will approach, or equal, that given by Eq 8.2.

A rough estimate of the percentage settlement to be expected in the side fills may be obtained from Fig. 8.24, discussed in Sec. 8.19.

8.5. Wall Thickness to Resist Earth Load

The bending-moment, or flexural-stress, method of analysis does not always apply to steel pipe under earth loads. Almost all analyses of steel conduits have been directed at the structure itself and the load it carries. This report presents two methods of design, both of which use the elastic-ring theory for analyzing the pipe barrel; both are based on designing flexible steel pipe to obtain a required vertical deflection, and both recognize earth assistance at the side of the pipe. The first method is by Spangler (*2, 3*), who uses a single selective constant for the modulus of passive resistance of the earth. The second is by Barnard (*4*), who analyzes the pipe barrel both for elastic-ring strength and for confined-arch strength,

determines the outward thrust of the pipe wall, and then considers the passive deformation of the soil at the side of the pipe under this load to be a "sidewise" settlement and employs the principles of soil mechanics to determine its magnitude. In the Spangler formula, it is very difficult to determine for use the proper value of the modulus of passive resistance of the enveloping soil. The Barnard method permits use of either typical values of engineering properties of soils or any specially determined values in a given case.

8.6. Design Factors in Spangler Formula

The formula for the design of *flexible* pipe culverts evolved by Spangler is:

$$\Delta x = D_l\left(\frac{KWr^3}{EI + 0.061er^4}\right) \ldots\ldots (8.3)$$

in which:

Δx = horizontal deflection of flexible pipe (in.)
K = bedding constant (0.10 for flat-bottom trench with untamped backfill or with backfill tamped to horizontal diameter or spring line of pipe)
W = vertical load per unit of pipe length (lb/lin in. of pipe)
r = radius of pipe (in.)
E = modulus of elasticity of pipe metal (30,000,000 for steel)
I = moment of inertia of cross section of pipe wall (in.^{4}/lin in. of pipe)
e = modulus of passive resistance of enveloping earth (psi/in.)
D_l = deflection lag factor (1.25–1.50). (Note: Some water works authorities do not apply the lag factor to the deflection caused by live load.)

It is to be noted in Eq 8.3 that W is the *total* load on the conduit (in pounds per linear inch). Therefore, to obtain design loads W for *flexible* pipe in trenches using Eq 8.1 and

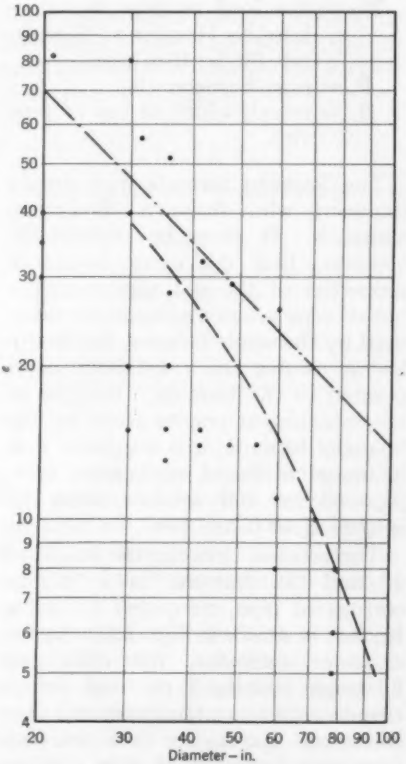

Fig. 8.6. Relationship Between Values of Spangler *e* and Pipe Diameter

The values of e, *calculated from measured test data, decreased as the pipe diameter increased* (7).

including both live and dead loads (and considering units of measurement):

$$W = \frac{1}{12}\left(W_e \frac{B_e}{B_d} + W_L\right) \ldots\ldots (8.4)$$

in which:

W = total load on flexible pipe (lb/lin in. of pipe)
W_e = dead load on pipe from Eq 8.1 (lb/lin ft of pipe)
W_L = live load on pipe as calculated by Marston's (*1*) methods (lb/lin ft of pipe)
B_e = pipe diameter (ft)
B_d = trench width at top of pipe (ft).

The Spangler formula may satisfy engineers who desire a theoretical approach. It must be understood, however, that the earth assistance properties of the soil surrounding a buried pipe are not adequately measured by the single factor *e*, the modulus of passive soil resistance, incorporated in the formula. Because of the inconsistent results given by the Spangler formula, it is suggested that its range of useful application is to pipe 60 in. and smaller under fill heights of 10 ft and less.

The relation between the height of fill and the diameter of a 16-gage corrugated pipe, computed (*5*) using Eq 8.3, is shown in Fig. 8.5. As the diameter increases, the calculated fill height causing 1 per cent deflection decreases to a minimum and then increases. Experience does not confirm such behavior of pipe. When Eq 8.3 was used to determine the thickness of a pipe 156 in. in diameter to deflect 2½ in. under a 20-ft backfill cover with $e = 30$, the figure for required wall thickness was negative.

8.6.1. *Earth modulus*, e. The factor *e* in the Spangler formula is intended to measure the passive resistance of the earth at the sides of the pipe on an elastic basis. It is supposedly dependent on the type of soil and the void ratio, or degree of compaction, in that soil. Spangler reports measured values of *e* in *embankments* (*2*) as given in Table 8.1. Note that tamping the side fills practically doubled the value. When calculating the value of *e* corresponding to the then available measured deflections for smooth steel water pipe of many different diameters, Barnard (*7*) found

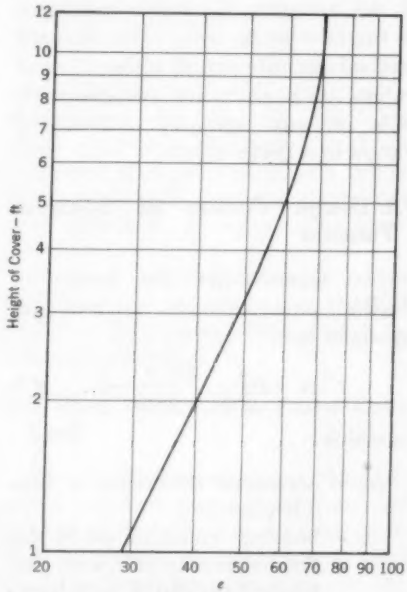

Fig. 8.7. Relationship Between Values of Spangler *e* and Height of Fill

Measured deflections showed that, for the same pipe, the value of e *increases as the fill height increases* (5, 6).

e to decrease as the pipe diameter increased (Fig. 8.6).*

When the measured deflections in a carefully conducted field test (*6*) were employed (*5*), to calculate the value of e using Eq 8.3, for increment increases in fill height on the same pipe, the resulting value of e varied from less than 30 at 1 ft of cover to about 70 at 12 ft of cover (Fig. 8.7).

The modulus e has been found consistent with measured deflection and with soil properties *only* in degrees of soil compaction and loading seldom reached in pipe installations (*4*). Therefore, e must be treated as a selective constant whose actual value in a given case is unknown. A value of $e = 30$ appears to be an acceptable approximation for deflection calculations for pipe 60 in. and smaller under cover of 10 ft or less, for either trench or fill conditions.

Barnard (*8*), using the Spangler formula with $e = 30$ and load as in Eq 8.4, developed the deflection of 60-in. and smaller for steel pipe under external load in trenches, as shown in Table 8.2. These data seem to check quite well with experience in trenches of common depth and where ordinary backfilling methods have been used.

The method of backfilling very flexible pipe over 4 ft in diameter may completely offset the designer's calculations if care is not taken. Initially the pipe should have its full vertical diameter, at least. The backfilling should be built up evenly on each side at the same time. Backfilling from one side and then the other may be disastrous. Ordinary tamping up to the horizontal diameter, plus careful backfilling above that point, will insure installations wherein the vertical deflection is normal.

TABLE 8.1

Values of Spangler e *Determined by Experiment*

Expt. No.	Kind of Soil	e psi/in.
1	Black silty loam (not compacted)	14
2	Well graded gravel (not compacted)	32
3	Yellow sandy clay loam (not compacted)	13
4	Yellow sandy clay loam (tamped, dry)	27

8.6.2. *Deflection*, Δx. Strictly speaking, Δx is the horizontal deflection of the pipe, but, within the range of use, experience has shown that it may be considered to be the vertical deflection as well.

The amount of vertical deflection that will cause collapse of flexible pipe is about 20 per cent of the nominal diameter. Structurally speaking, therefore, deflections up to 10 per cent of diameter are amply safe. The spun coal-tar enamel meeting AWWA standards will safely take a 10 per cent deflection. Corrugated culvert installations are commonly designed using 5 per cent deflection.

* In 1957, after a study in similitude,[7a] Spangler modified the denominator of his original formula (Eq. 8.3) by a change in nomenclature which substitutes $e'r^3$ for er^4, in which $e' = er$. He then believed that there is a size effect not originally recognized and that it is more nearly correct to consider the modulus of soil reaction e' to be a constant. The actual value of e' for a specific instance is still unknown, but a tentative value of $e' = 700$ is suggested by Spangler for side fill soil which has been compacted to 90 per cent or more Proctor density. The corresponding value of e for different pipe diameters when $e' = 700$ is shown as the dotted-dashed line in Fig. 8.6. The comparison is clear between these theoretical values of e and values calculated by Barnard from measured data on steel water lines, as shown by the various dots.

TABLE 8.2

*Deflection of Steel Pipe Under External Load in Trenches**

1	2	3	4	5	6	7	8
Nominal Size *in.*	2% of Diam. *in.*	Wall Thickness—*in.*		*d* = Deflection—*in.*			
				5-ft Cover		10-ft Cover	
		Fraction	Decimal	No Truck	With Truck	No Truck	With Truck
6	0.12	1/8	0.1250	0.01	0.02	0.02	0.02
		3/16	0.1875	0.00	0.01	0.01	0.01
8	0.16	1/8	0.1250	0.03	0.07	0.06	0.07
		3/16	0.1875	0.01	0.02	0.02	0.02
10	0.20	1/8	0.1250	0.07	0.17	0.15	0.18
		3/16	0.1875	0.03	0.06	0.05	0.07
12	0.24	1/8	0.1250	0.15	0.32	0.27	0.33
		3/16	0.1875	0.06	0.12	0.10	0.12
		1/4	0.2500	0.03	0.06	0.05	0.06
14	0.28	1/8	0.1250	0.21	0.48	0.42	0.52
		3/16	0.1875	0.09	0.21	0.19	0.24
		1/4	0.2500	0.04	0.10	0.09	0.11
16	0.32	1/8	0.1250	0.30	0.60	0.56	0.68
		3/16	0.1875	0.15	0.31	0.28	0.35
		1/4	0.2500	0.08	0.16	0.14	0.18
18	0.36	3/16	0.1875	0.21	0.41	0.39	0.47
		1/4	0.2500	0.12	0.22	0.21	0.26
		5/16	0.3125	0.07	0.13	0.13	0.15
20	0.40	3/16	0.1875	0.28	0.53	0.49	0.59
		1/4	0.2500	0.17	0.31	0.29	0.35
		5/16	0.3125	0.10	0.19	0.18	0.21
24	0.48	3/16	0.1875	0.41	0.75	0.71	0.83
		1/4	0.2500	0.29	0.54	0.50	0.59
		3/8	0.3750	0.14	0.24	0.23	0.27
30	0.60	1/4	0.2500	0.40	0.71	0.70	0.83
		5/16	0.3125	0.31	0.54	0.54	0.64
		3/8	0.3750	0.23	0.40	0.40	0.48
36	0.72	1/4	0.2500	0.48	0.83	0.87	1.02
		5/16	0.3125	0.41	0.71	0.75	0.87
		3/8	0.3750	0.34	0.58	0.62	0.72
42	0.84	1/4	0.2500	0.53	0.98	0.95	1.14
		3/8	0.3750	0.44	0.80	0.78	0.92
		1/2	0.5000	0.32	0.57	0.57	0.68
48	0.96	5/16	0.3125	0.52	0.87	0.94	1.09
		3/8	0.3750	0.48	0.81	0.87	1.01
		7/16	0.4375	0.43	0.73	0.78	0.91
		1/2	0.5000	0.38	0.65	0.70	0.81
60	1.20	3/8	0.3750	0.57	0.98	1.05	1.22
		1/2	0.5000	0.51	0.87	0.95	1.10

* Trench width equals outside diameter of pipe plus 2 ft. Field conditions: flat-bottomed trench, tamped backfill.

From Barnard (8)

For steel water pipe, permitting 2 per cent deflection is good practice, although for large-diameter pipe, unlined, deflection has sometimes amounted to as much as 10 per cent on well engineered lines.

Permissible deflection for cement-lined pipe is on the order of 2 per cent.

The effect on hydraulic efficiency of a pipe deflected by as much as 5 per cent is entirely negligible as its area is 99.75 per cent of that of a perfect circle and its hydraulic radius is also 99.75 per cent. When the deflection is 2 per cent, the corresponding figure is 99.95 per cent for both area and hydraulic radius.

8.6.3. *Bedding constant*, K. For field conditions usually encountered in steel pipe installations, a good design value of K is 0.10. The range of K is from 0.110 for point support to 0.083 for bedding the full width of the pipe.

8.6.4. *Design load*, W. Experience indicates that the calculated deflection will be more nearly accurate if W approaches the minimum value for smaller pipe given by Eq 8.2 and the maximum value for larger pipe given by Eq 8.1. In this sense, large pipe is 48-in. and over.

8.6.5. *Deflection lag factor*, D_l. It has been observed that corrugated-pipe culverts continue to deflect slowly for a period of time after installation. The gradual increase in deflection is an action somewhat similar to foundation settlement and time consolidation of soils. The increase becomes slower and slower, however, and after a period of years virtual stability is attained. Factor D_l relates initial deflection to final deflection. Values observed by Spangler range from 1.38 to 1.46. A design value suggestion is 1.25 to 1.50. Some water works authorities do not

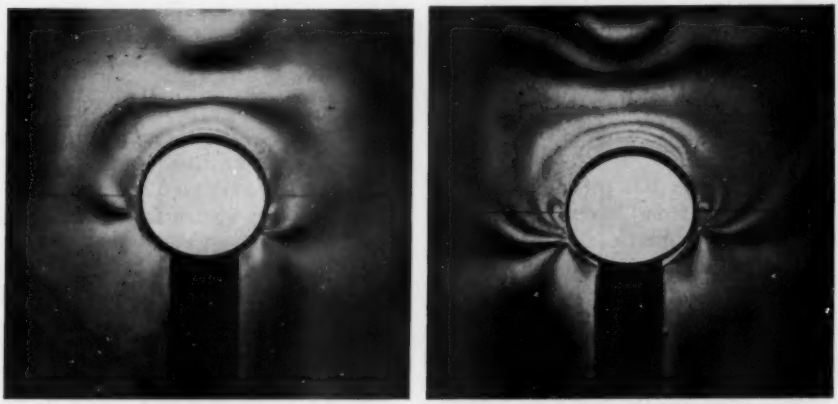

Fig. 8.8. Fringe Pattern Around Ring on Slip-Fit and Over-Size Holes

In a first series of tests, the hole size was a slip-fit for the ring. The fringe pattern (at the left) indicates a relatively uniform radial stress. The stress pattern in a second series of tests, when the diameter of the hole was $\frac{1}{32}$ in. larger than that of the ring, is shown at the right (4).

apply the lag factor to the deflection caused by live load.

8.7. Barnard's Sidewise Settlement Theory

A footing resting on a ground surface settles under a vertical load. An initial settlement occurs immediately, and usually a further and slower settlement occurs as time passes. The extent of the settlements depends upon the load intensity, the number and nature of applications, the size of the footing, and the initial and final engineering properties of the soil assuming the load. To this extent there is interaction between the soil and every manmade structure resting on it.

The engineer has means of designing footings from knowledge of the engineering properties of the materials in the footings. Also at his disposal are means of estimating soil settlement when he knows the engineering properties of the soils.

A rigorous analysis of the problem of underground conduits will probably remain impractical as long as there are unmeasured variables in field installation, in backfilling and compacting, and in precise knowledge of soil engineering characteristics. Under such circumstances, extreme accuracy is not expected. The semiempirical approach is, however, justified as a practical approximation.

Experience shows that the vertical diameter of a buried *flexible* steel pipe under dead load and live load decreases and that the horizontal diameter increases. The vertical loads obviously result in horizontal thrusting loads against the soil at the sides of the pipe. Such thrusts have been measured. Postulating that this horizontal side thrust action resembles vertical thrust action on a footing and that the resulting soil deformation at the pipe side is a "sidewise" instead of a vertical settlement, Barnard (*4*) determines this sidewise settlement using empirically the principles established for estimating vertical settlement under foundations. The distance the pipe side wall moves against the soil is termed "passive deformation" of the soil. The vertical change in diameter or deflection

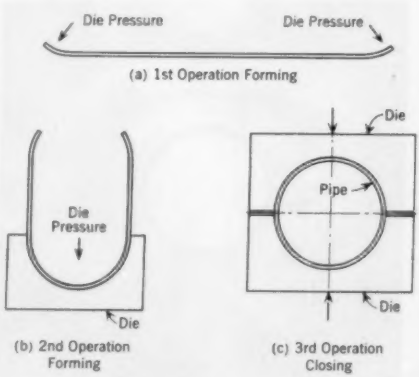

Fig. 8.9. Operations in Forming High-Pressure Steel Pipe

Operations shown are performed on pipe to 36-in. size having a wall thickness as small as 0.25 in. (4).

of the pipe is two times the passive deformation. The true engineering properties of both the steel pipe and the earth envelope enter directly into these calculations for deflection. The method of calculation has checked closely with measured deflections under low, medium, and high cover. The calculation process is simple and principally graphical. The method has the distinct advantage of telling directly the amount of vertical ellipsing stulling, or strutting necessary

in a given instance to avoid exceeding the final permissible maximum pipe deflection. This advantage is due to the fact that the calculated sidewise settlement is really a distance measurement, even though it is expressed in terms of pipe diameter.

8.8. Pipe Action

When there is little or no side support, a steel pipe underground functions as an elastic ring loaded at top and bottom. Relatively thick rings carry more load with less deflection than do thinner rings and may be said to possess greater "ring strength." In a given case, however, if the ring strength is not sufficient to carry the full load, the pipe deflects and soil side support builds up. Considering progressively pipe having less and less ring strength, it is seen that the resisting action in the different walls must change from that of a ring loaded at top and bottom to the arch action of a totally enclosed ring. In the latter case, the confined ring must possess sufficient arch strength to resist the load adequately even though its ring strength may be inadequate. Data on ring strength are given in Sec. 8.12.

8.9. Arch Strength

That a confined thin steel shell under outside load must act in compression with bending resistance relatively unimportant is proved by wide experience (*4, 9*). Two outstanding examples of culvert performance will serve to demonstrate the tremendous resistance to earth pressure possessed by thin metal cylinders. (The fact that corrugated pipe is involved in these instances does not make them less relevant.)

A 72-in. diameter, 8-gage (0.172-in.) corrugated metal pipe under the "hot track" at a steel mill is one outstanding example. This culvert carries one of the heaviest wheel loads known—343 tons on 16 wheels—88.5 tons of which can act on the culvert when the wheel truck is directly over it. The cover of earth, which is only 3.5 ft, imposes an especially critical condition.

When analyzed for flexural stress by the classic methods, the indicated maximum fiber stress is 972,000 psi. Actually, the culvert is made of soft iron with a maximum specified tensile strength of 44,000 psi and a yield strength of 27,500 psi. The calculated fiber stress is therefore 22 times the ultimate and 35 times the yield strength. In spite of the figures, however, experience demonstrates that, as a culvert, this structure has a safety factor of four or five. The unit arch stress is approximately 6,500 psi. Careful observations showed no change during the first 9 years with a load passing over it several times daily. Now, after 29 years (1958), it is still in service with no appreciable change in recent years.

Another outstanding example is a 60-in. diameter, 8-gage corrugated culvert under a 77-ft railroad fill. If the classical bending-moment formula analysis and dead-load assumptions sometimes used by engineers are employed, and if any effect of train live load is ignored, the indicated maximum fiber stress is 1,100,000 psi—25 times the ultimate. Investigations, however, showed that no appreciable change in shape has occurred in 11 years. The unit arch stress is approximately 9,700 psi.

The validity of the statement concerning thin steel shell under outside load can be demonstrated by labora-

tory test and by the manufacturing procedure of forming steel pipe in a die.

8.9.1. *Proof by laboratory test.* An endeavor was made to demonstrate photoelastically that the postulation of load distribution around embedded steel pipe used in the Barnard theory was not seriously in error. A hole was drilled in a $\frac{3}{8}$-in. thick plastic plate to accommodate an aluminum ring 1.25 in. in diameter, 0.034 in. thick, and $\frac{1}{2}$ in. long. A slot $\frac{5}{8}$ in. wide extended from the hole to the top of the plate. A steel loading plunger $\frac{5}{8}$ in. wide operated in this slot. The contact face of the plunger was ground to fit on the tube. It delivered a load covering one-half the tube diameter.

In the first series of tests, the hole size was a slip-fit for the ring. This would come as near developing uniform radial pressure in the ring as could be expected, considering the elements and factors involved in the test. The fringe pattern (Fig. 8.8) indicates a relatively uniform radial stress except near the area of application of a concentrated load, where a different pattern would naturally be expected.

In the second series of tests, the diameter of the hole was made $\frac{1}{32}$ in. larger than that of the ring. Thus, the ring was allowed to deflect about

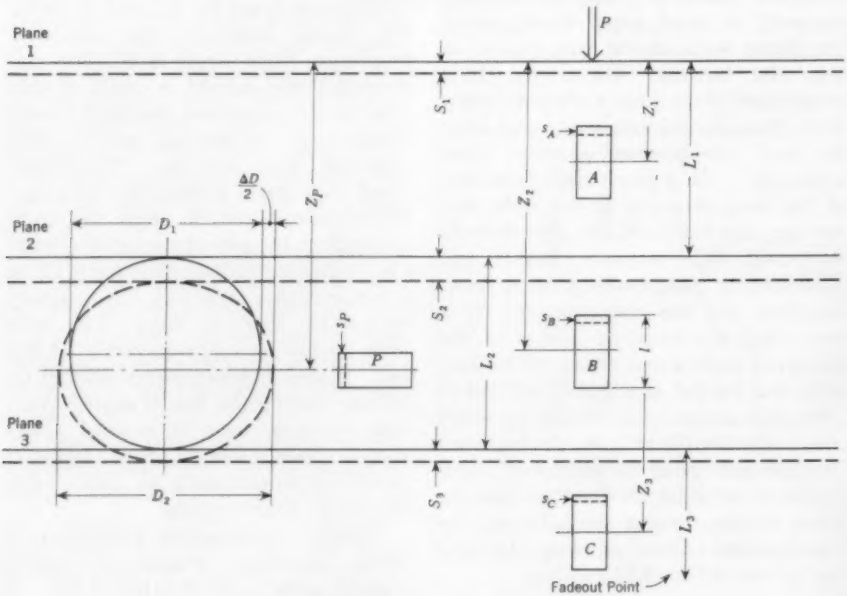

Fig. 8.10. Soil Settlement Under Load

The diagram pertains to settlement of fill and deflection of flexible pipe. See Sec. 8.10 for an explanation of letter symbols and a discussion of settlement (4).

2½ per cent vertically before full contact was established in the horizontal direction. The idea here was to develop, at the extremities of the horizontal diameter, a pattern which would resemble the familiar onionlike bulb of pressure existing under footings delivering load to soil. Figure 8.8 shows the stress pattern achieved. To be noted are the beginnings of the "ears," which, when full grown, are postulated as being similar to that "bulb of pressure" under a footing load except for having been rotated 90 deg from the vertical and converted to a horizontal thrust.

Noteworthy are the calculated stresses in the model ring. As an elastic ring without side support, placed between the heads of the testing machine, it held 30 lb with a deflection of about 2½ per cent. The maximum load in the first series of tests was 200 lb. In the second series the maximum was 250 lb. The stress determined, using the method for "arch load"—see Paragraph 8.11(c)—is 5,900 psi for the 200-lb load and 7,300 psi for the 250-lb load.

Calculated on the basis of section modulus resistance, the apparent stress in the ring under the plunger load is about 3,750,000 psi for the 200-lb load and 4,700,000 psi for the 250-lb load.

The material in the ring had a yield strength of about 55,000 psi and an ultimate of about 75,000 psi. The ring used was not harmed. After being removed from the testing machine, it rolled on plate glass without showing a balance point.

8.9.2. *Proof by die-forming pipe.* The process of manufacturing high-test line pipe furnishes continuous test proof of the tremendous arch loads which can be carried by smooth-wall steel pipe. The forming operations are shown in Fig. 8.9. The pipe is made from steel plates having a length equal to the finished pipe section and a width somewhat greater than the pipe circumference. The first operation consists of die-forming the ends of the plate to a radius approximating that of the finished pipe

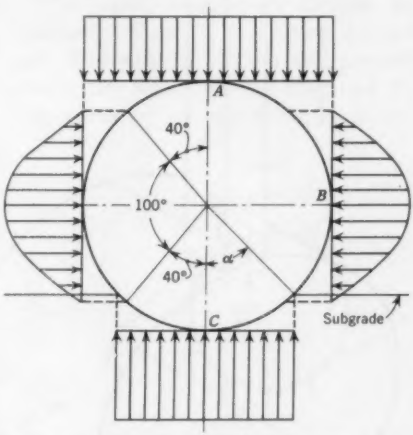

Fig. 8.11. Distribution of Load and Thrust

The diagram gives an idea of the distribution of the horizontal thrust of the pipe side against the soil. A, B, *and* C *are reference points;* α *is one-half the bedding contact angle* (4).

as shown in Fig. 8.9a. The second operation is performed in a U-shaped die, with the result shown in Fig. 8.9b. In the last operation, as shown in Fig. 8.9c, two semicircular die sections are brought against the U-shaped pipe, forcing it into a complete circle, with sufficient pressure between the plate edges to permit electric-resistance flash welding. The

final closing of the die forces weld metal from the welded joint inward toward the center of the pipe and outward at the top. Such operations are performed on 30-in. or 36-in. pipe having a wall thickness of as little as $\frac{1}{4}$ in. Naturally, the pipe sections endure this operation successfully or the method of manufacture would be discarded. It is necessary to recognize the important part that the engineering properties of the pipe

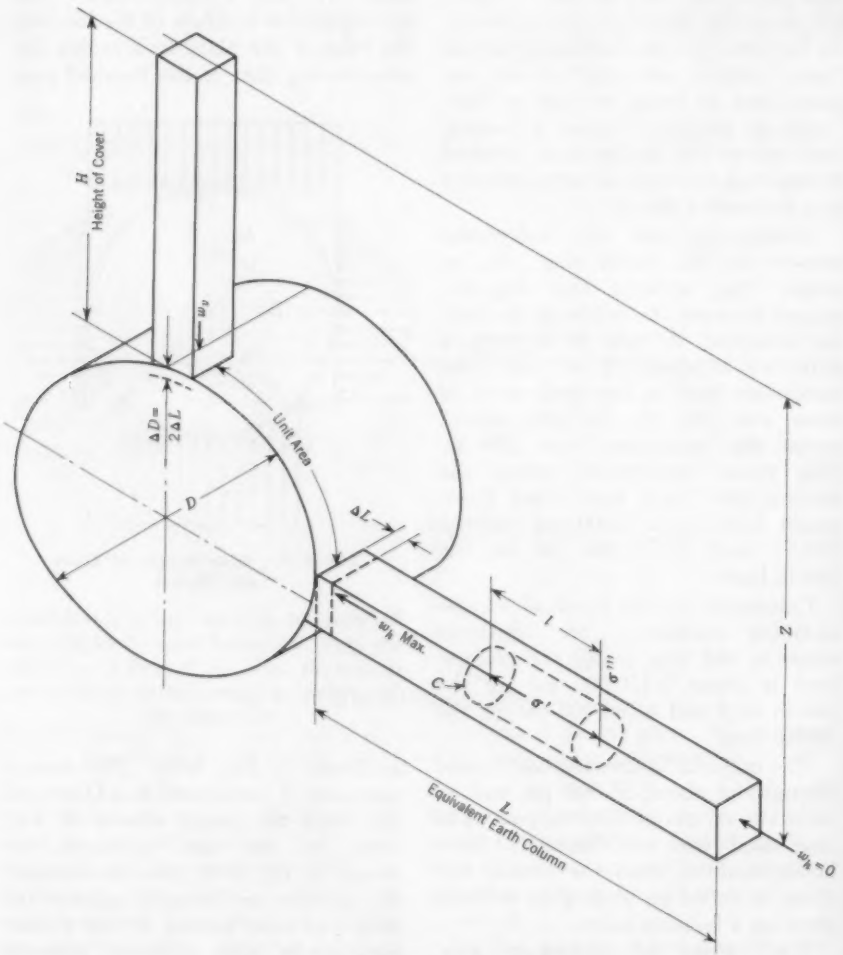

Fig. 8.12. Equivalent Earth Column

The diagram shows the basic concept of the equivalent-earth-column method of calculation. See Sec. 8.13 for an explanation of letter symbols (4).

material and the die material have in this manufacturing operation.

8.10. Barnard's General Theory

The soil which surrounds a pipe in a trench or in a fill is a remolded soil. Assume that it is required to determine the settlement under load P, Fig. 8.10, in a fill made of two kinds of materials and built on a natural ground at Plane 3. There are several ways to do this. One method is to make properly controlled triaxial tests of the soils involved, to determine their engineering properties (*11*). That method is used in this report for illustrative purposes because the data on several typical remolded soils were available (*12*). Settlement may be determined by void ratio-pressure data on soil samples or, in more approximate form, by using data developed by Proctor (*13*). The latter subject is further discussed in Sec. 8.19.

In Fig. 8.10, the load distribution in the soil due to load P is found by the methods of Boussinesq, and a "fadeout" point is established for calculation purposes. When triaxial Test Specimen A is tested with confining pressure equivalent to the height of earth column Z_1, and axial pressure reaching the specimen location, the ratio of its shortening, s_A, to its length, l, may be considered the same as the ratio of settlement S_1 to the thickness of layer L_1, which it represents. If s_A is expressed as a percentage of l, then S_1 is the same percentage of L_1.

The same reasoning applies to Specimens B and C and layers L_2 and L_3. The sum of all the settlements equals the settlement of the load P. The accuracy of the findings is influenced by the many uncertainties of sampling, testing, and calculations of underground load distribution.

The foregoing calculation is an estimate of initial settlement. Settlement due to time consolidation is additive. In this report, time consolidation is accounted for by a "lag factor" and is discussed in Sec. 8.18.

The left-hand portion of Fig. 8.10 indicates a steel pipe as it might be installed in a fill such as considered before. The pipe is shown as having settled with Plane 3 and deflected

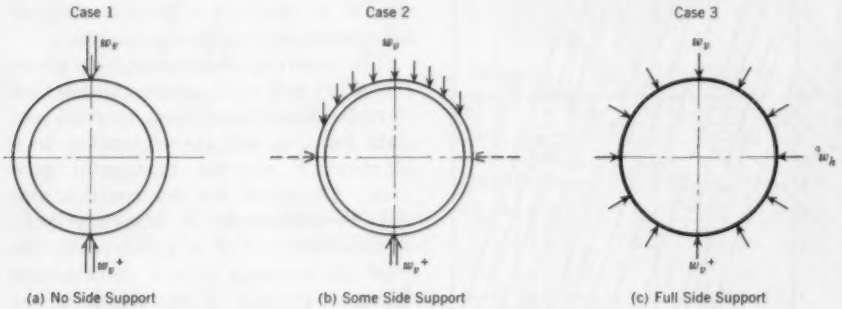

Fig. 8.13. Cases of Conduits and Loadings

As explained in Sec. 8.13, w_v *represents the total vertical unit load on pipe (psi); and* w_h, *the unit horizontal load (psi) transfer load contact pressure* (4).

enough to occupy the space between Planes 2 and 3 after initial settlement. For purposes of engineering design, it is postulated that the pipe is supporting the full weight of the prism of earth above it because there are no "friction planes" (see Sec. 8.3) to modify the load. For a flexible pipe

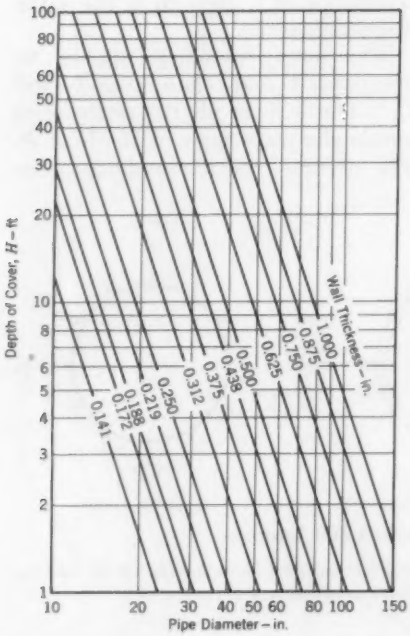

Fig. 8.14. Theoretical Height of Cover "Ring Load" for Steel Pipe With 2 Per Cent Deflection

The weight of the soil was 125 lb/cu ft (4).

this is the worst situation possible, so it can be used for calculation purposes.

The horizontal thrust of the pipe side against the soil is assumed to be distributed in some such fashion as indicated in Fig. 8.11. As the total of all forces must be in equilibrium, the force of maximum intensity on the horizontal diameter must exceed the intensity on the vertical diameter by some amount. It is affected also by the instantaneous radius of the pipe in contact with the soil at a given point, such as *A*, *B*, or *C*. This is discussed in Sec. 8.15.

The size of a footing is a factor in its settlement, and therefore the diameter of a pipe is a factor in its "sidewise" settlement. The greater the size or diameter, the greater the settlement for given load intensity in unit per area. The settlement or passive deformation at the pipe side may be expressed in terms of the pipe diameter.

The loading at the side of a pipe is similar to that for a flexible strip of infinite length uniformly loaded.

The passive deformation for a given remolded soil with given compaction may be determined from triaxial test data for that soil corresponding to a location *P* on the horizontal pipe axis. Of course, the deformation also may be determined by other methods, if available. For a given soil, the load to cause a given deformation becomes greater as the depth of the soil below the surface becomes greater. This is important in determining pipe deflection.

The basic concept of the equivalent-earth-column method of calculation is shown in Fig. 8.12. The engineering properties of the soil are determined for a position on the horizontal pipe axis.

8.11. Definition of Terms

Further discussion will be facilitated by defining and using certain terms:

a. Ring load—the load carried by the conduit shell as determined by moment-modulus calculations for ring diameter changes of about 5 per cent or less.

b. Ring aid—the horizontal thrust toward the pipe due to active earth pressure; it may be calculated on the basis of one-third vertical pressure for sand and one-half for clay; *it is used only to reduce ring load, not arch load.*

c. Arch load—the load carried by the conduit shell under external, radially acting pressure determined by using the Barlow formula:

$$2St = pD \quad \ldots\ldots\ldots\ldots (8.5)$$

or:

$$St = pR \quad \ldots\ldots\ldots\ldots (8.6)$$

in which:

S = stress (psi)
t = wall thickness (in.)
p = pressure (psi) due to height of cover above pipe
D = conduit diameter (in.)
R = instantaneous radius of deformed conduit at given point (in.).

The indicated stress in the pipe wall from Eq 8.5 and 8.6 is, respectively:

$$S = \frac{pD}{2t} \quad \ldots\ldots\ldots\ldots (8.7)$$

or:

$$S = \frac{pR}{t} \quad \ldots\ldots\ldots\ldots (8.8)$$

d. Transfer load—the load in excess of the ring load which is imparted to the soil at the sides of the pipe.

e. Passive deformation—the horizontal "settlement" or distance the soil is compressed by the transfer load.

f. Contact pressure—the measure of the force acting between metal surface and soil surface in an approximately horizontal direction.

8.12. Ring Strength and Arch Strength

Three different cases of pipe resisting action are recognized:

Case 1. In this case (Fig. 8.13a), the wall thickness and diameter of the pipe selected to meet pressure and other service requirements are such that its ring strength can carry satisfactorily all earth loads upon it without undue deflection. This case is quite frequent in water works practice, and no problem occurs in installation. Ring strength for 2 per cent deflection is given in Fig. 8.14.

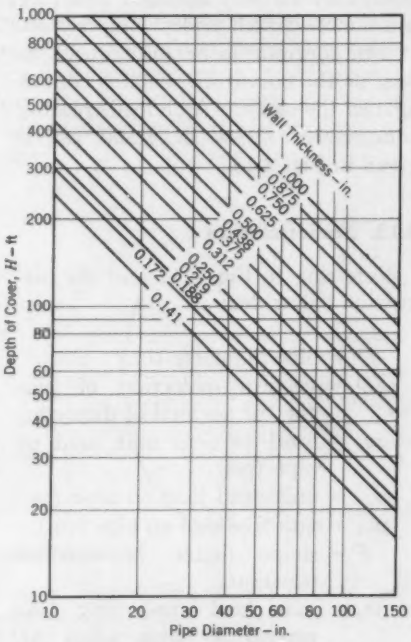

Fig. 8.15. Theoretical Height of Cover "Arch Load" for Steel Pipe With Full Side Support

The stress (S) *was equal to 7,500 psi* (*Eq. 8.5*); *the weight of the soil was 125 lb/cu ft* (4).

Case 2. In Case 2 (Fig. 8.13b), the pipe ring strength is sufficient to carry part of the earth load and live load, but not all of it, without undue deflection. Some side support must be afforded by the earth.

Case 3. In Case 3 (Fig. 8.13c), the pipe is a completely flexible thin ring and by itself can carry very little, if any, of the live and dead loads without undue deflection. Full mobilization of earth assistance is needed to retain the degree of roundness required. The enveloping forces are essentially radially acting. The ring is a confined arch under compression. It will continue to act as an arch as long as the curvature remains convex against the earth. Arch load creating compressive stress of 7,500 psi is given in Fig. 8.15.

8.13. Nomenclature

Referring to Fig. 8.12 and the discussion below, let:

D = pipe diameter (in.)
ΔD = vertical deflection of pipe (in., or decimal of diameter)
w_v = total vertical unit load on pipe (psi)
w_{vd} = unit dead load on pipe (psi)
w_{vl} = unit live load on pipe (psi)
F = shape factor (a selective constant)
w_p = unit load (psi) ring load carried by pipe when $\Delta D = 0.02D$ (values from Fig. 8.14)
w_a = unit load (psi) ring aid afforded by active soil pressure
w_h = unit horizontal load (psi) transfer load contact pressure
L = effective length of equivalent earth column (in., or in terms of D)
ΔL = passive deformation (in., or decimal of L)
ϵ = strain in soil (in./in., or decimal of tested length) under load of contact pressure intensity, w_h
ϵ_{w_h} = strain (per cent) corresponding to given w_h in Fig. 8.18–8.24.

8.14. Intensity of Vertical Load

Total vertical load equals dead load for height of cover, H, plus live load, impact, and so forth:

$$w_v = w_{vd} + w_{vl} \quad \text{(8.9)}$$

When a trench is backfilled by ordinary methods and without tamping, some portion of the active pressure of the soil in the backfill acts against the side of the pipe and tends to support it. For this case,

$$w_v < w_p + w_a; \quad w_v - w_a < w_p \quad \text{(8.10)}$$

If w_p is not greater than $w_v - w_a$, Eq 8.10 should be ignored altogether.

8.15. Intensity of Contact Pressure

Intensity of contact pressure between pipe and soil is dependent on: [1] the vertical load, [2] the degree of stiffness of the pipe as defined in Sec. 8.12, and [3] the instantaneous radius of the point of contact considered—in this case, the extremity of the horizontal axis.

A flexible steel ring or pipe without appreciable ring strength or stiffness (Case 3, Sec. 8.12) is best thought of as being a stress gage itself when confined by an earth envelope and supporting earth loads. If the pipe is perfectly round, there is no bending moment and the compressive stress in the wall is given by Eq 8.8:

$$S = \frac{pR}{t}$$

It is obvious that the same stress must prevail in every portion of the pipe periphery. The vertical load initiates the stress. The confining pressure in the soil must maintain that same stress at every point or the pipe will change shape. Therefore, in a truly round pipe, the confining pressure must act radially and be equal to p in Eq 8.8.

Also, it is seen in Eq 8.8 that, for a given stress and thickness, the value of p must increase if R decreases and decrease if R increases. Therefore, to maintain this circumferential compressive force equally at all points after a round pipe has deflected, the outside radially acting confining force, or earth pressure, must rise at points of lesser radius.

In an initially round pipe of diameter D, $a = b$ where a and b are located as shown in Fig. 8.16. As the pipe deflects, the amount by which b is lessened is added to a. The unit vertical load on a flattened pipe creates in the wall at the vertical diameter a tangential compression stress given by Eq 8.8, with R equal to the instantaneous radius which occurs at the vertical axis of an assumed ellipse. This radius is larger than that of the original pipe and creates a higher stress in the pipe wall. Conversely, the instantaneous radius at the extremity of the horizontal axis is smaller than that of the original pipe and, as the same stress in the wall must be maintained there to prevent pipe wall movement, the contact pressure on the horizontal axis must be greater than that on the vertical axis.

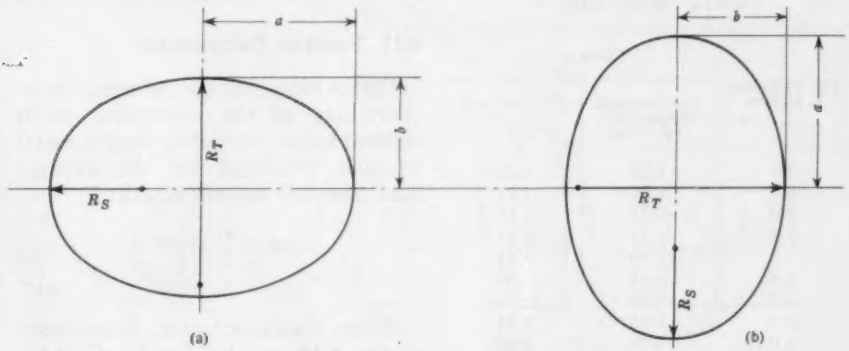

Fig. 8.16. Instantaneous Radiuses

Shown are the radiuses of round pipe expanding horizontally (a) and ellipsed for installation (b). (Figure from Ref. 4.)

The calculation constant which accounts for this condition when combined with the figure for length of equivalent earth column is here termed the "shape factor." It is further discussed in Sec. 8.17.

The intensity of contact pressure covering Cases 1 and 2 in Sec. 8.12 can now be stated as:

$$w_h = F(w_v - w_p) \quad \ldots\ldots (8.11)$$

For Case 3, $w_p = 0$.

8.16. Length of Equivalent Earth Column

To determine the length of equivalent earth column, a "footing width" must first be assumed. This "width" is taken as a chord subtended by a central angle of 100 deg, as shown in Fig. 8.17. Dimension B is the usual one-half footing width used in determining vertical stress at selected depths below that footing. In Fig. 8.17, Line I represents the influence line for calculating vertical stress under a strip when the length-to-width ratio lies between 2 and infinity, as shown by Terzaghi (*15*). The stress central in each increment of B is shown in the corresponding blocks. Ten per cent of stress is yet unaccounted for in the ninth block from the soil-pipe contact line.

Line II represents the hypotenuse of a right-angle triangle which contains the same area (between line and horizontal axis) as that under Line I, except that the missing 10 per cent stress has been added. On this basis, the horizontal leg of the triangle has a length of approximately 6.5 times B. This distance from the pipe face is therefore considered the point of practical "fadeout" *for calculation purposes.* Thus, in Fig. 8.17,

$$B = r \sin 50° = 0.766r$$

Or, in terms of pipe diameter D:

$$B = 0.383D$$

The length, L, of the equivalent earth column is:

$$L = 6.5(0.383)D$$

$$L = 2.49D \text{ (say, } 2.5D) \quad (8.12)$$

TABLE 8.3

Values of Shape Factor, F

Pipe Deflection % *of diam.*	Values of F*	
	Pipe Deflected Horizontally (Fig. 8.16a)	Pipe Strutted Vertically (Fig. 8.16b)
0.0	1.25	1.25
0.5	1.29	1.21
1.0	1.33	1.18
1.5	1.37	1.14
2.0	1.40	1.11
2.5	1.45	1.08
3.0	1.50	1.04
3.5	1.55	1.01
4.0	1.60	0.98
4.5	1.65	0.95
5.0	1.70	0.93

* To nearest 0.05.

8.17. Passive Deformation

On the basis that the deformation or shortening of the equivalent earth column under the fading load is equal to that produced by the average load over full column length:

$$\Delta L = \frac{w_h}{2}(2.5D)\epsilon$$

$$\Delta L = 1.25 w_h D \epsilon \quad (8.13)$$

When the shape factor, F, discussed in Sec. 8.15, is substituted in Eq 8.13:

$$\Delta L = F w_h D \epsilon \quad (8.14)$$

in which F equals 1.25 times a selective constant which depends on the ratio of instantaneous radiuses at the top and sides of the pipe.

The values of F for use in Eq 8.14, including the constant 1.25 in Eq 8.13, are given in Table 8.3 for round pipe which has been *decreased* in vertical diameter and also for pipe which has been *increased* or ellipsed in vertical diameter for construction purposes. For pipe normally installed round and allowed to deflect under load, the horizontal thrust *increases* as the deflection increases.

When pipe is formed, strutted, or stulled to *increase* its vertical diameter, *and it remains that way after installation*, the horizontal thrust *decreases* as the percentage of strutting increases. The stress in the pipe wall is less than for round pipe because the instantaneous radius of the force-activating top is less than that on the horizontal axis. Therefore, a lower balancing contact pressure is needed on the horizontal axis than on the vertical axis. This ellipsing cannot be carried too far, however, or the *active* pressure at the pipe side may reverse the convexity of the wall. Equation 8.8 may be used to investigate the latter condition with allowable stress held well below the yield point. The ring strength must be considered.

An important factor often overlooked in the case of strutted pipe is that the area of an ellipse is less than that of a circle with the same circumference. Therefore, the vertical load causing flattening of a strutted flexible pipe tends actually to make the pipe occupy a larger space in the soil as it expands horizontally and becomes more nearly round. This action causes a consolidation of the earth envelope and builds up pressure by changing the void ratio until the pipe becomes fully round. A round pipe, when deflecting vertically, actually occupies less and less volume as the deflection increases and must develop side support without the special aid supplied to the strutted pipe.

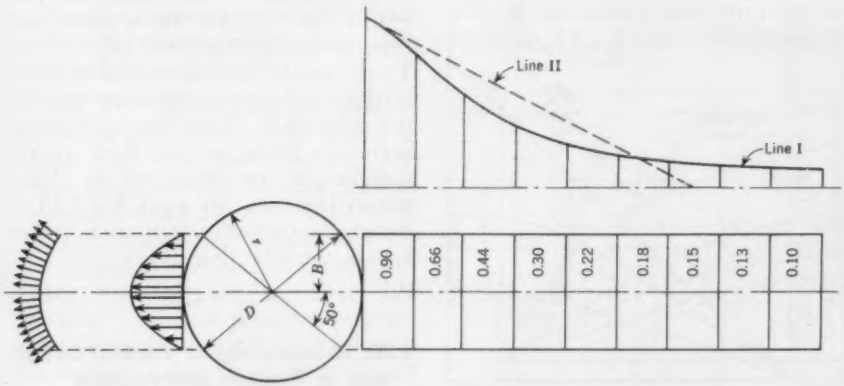

Fig. 8.17. Determination of Length of Equivalent Earth Column

See Sec. 8.16 for discussion of method. (Figure from Ref. 4.)

8.18. Time Lag Factor

The passive deformation determined using Eq 8.14 is the *initial* sidewise settlement. As with vertical settlements, experience shows that the sidewise settlements increase slowly with time. The increase becomes slower and slower, however, until, after a period of years, virtual stability is attained. In a particular case when engineering properties of a given soil are known, the time consolidation rate can be determined. Initial and

subsequent measurements made on actual pipe installations (*2*) show an increase with time of from 1.38 to 1.46 times initial deflection. A design value of 1.25 to 1.50 is suggested for average conditions.

8.19. Passive Deformation by Use of Diagrams

Figures 8.18–8.23 are included as aids in design and deflection control

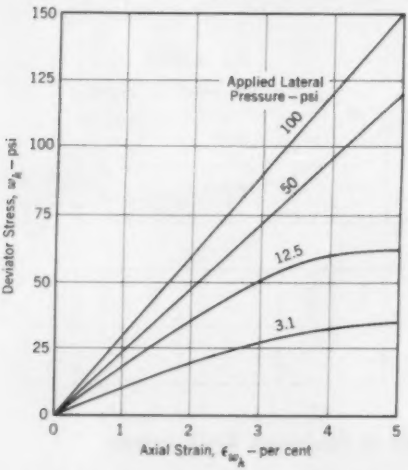

Fig. 8.18. Axial Strain in Typical Soil: Well Graded Sand

Data are from Test 1-T, Ref. 12.

calculations. The information has been selected from published data (*12, 16*). Other data are available. Those selected are more or less typical of remolded soils. If not otherwise indicated in the figures, soil specimens have been compacted to 98 Proctor density.

Figure 8.24 has been included for use by those designers familiar with Proctor's methods (*13, 14*) of soil compaction control. The diagram represents the results of about 900 soil specimen tests. The actual range of all varieties of the *remolded* soils tested indicates a spread of one-half to twice the values shown. The actual consolidation of *natural* soils in place in the 40–1,200-psi range of indicated saturation penetration resistance may be about one-third of the values shown in Fig. 8.24.

To use Fig. 8.18–8.24, calculate w_h using Eq. 8.11. Determine the value of z psi (Fig. 8.12). Enter at the left of the diagram for a given soil type, using the calculated value of w_h. Trace right to the intersection of a line (probably interpolated) representing the value of z. Read the axial strain vertically below in per cent, corresponding to the calculated w_h. Substitute this value for ϵ_{w_h} in Eq. 8.15 to determine passive deformation at the side of the pipe (Fig. 8.12):

$$\Delta L = \epsilon_{w_h} D \quad \ldots\ldots\ldots (8.15)$$

8.20. Relationship of Vertical Deflection to Passive Deformation

Because the vertical deflection of the pipe and the shortening of the earth column are now both expressed in terms of D, it becomes advantageous for calculation purposes to note the relationship. If ϵ is expressed as a percentage change in the length of the earth column, then double that percentage applies to the pipe deflection. In other words, if a given contact pressure in pounds per square inch causes a passive deformation corresponding to $\epsilon_{w_h} = 1$ per cent, the vertical deflection of the pipe will be 2 per cent of D. If the allowable vertical deflection is 5 per cent, the required passive resistance must be

mobilized with a passive deformation corresponding to $\epsilon_{w_h} = 2.5$ per cent.

8.21. Methods of Calculation

To facilitate calculation, unit pressures are expressed in pounds per square inch (psi). The following determinations are involved:

Step 1—intensity of vertical load (Eq 8.9)
Step 2—ring load (Fig. 8.14)
Step 3—ring aid (Paragraph 8.11b)
Step 4—arch load (Fig. 8.15)
Step 5—contact pressure (Eq 8.11)
Step 6—passive deformation (Eq 8.14 or 8.15).

Step 1. The load on the pipe, of course, is governed by the height of cover, plus live load, impact, and other factors. The vertical load is calculated using Eq 8.9. Unit earth loads in pounds per square inch for given heights of cover are shown in Table 8.4 for soil weighing 125 lb/cu ft. The unit pressure for other soil weights is in direct proportion to the tabular value for a given height of cover.

Intensity of truck live loads for the standard highway H-20 design and for the standard railroad locomotive Cooper E-72 loadings are shown in Table 8.5 for given heights of cover.

Steel pipe for water service is usually of such diameter and wall thickness that earth loading is not a problem. In the smaller sizes, the ring strength alone is sufficient to bear the load. When ring aid in the form of active earth pressure is added to the ring strength, sizes up to about a 36-in. diameter in most cases need no more than the ordinary attention given bedding and backfilling operations. But when the earth loads, including dead weight and live load, exceed the allowable ring load plus ring aid range, then from a design point of view the earth envelope becomes a part of the structure. Attention must therefore be given to the bedding and backfilling operations to the extent that sufficient strength is built into the backfill to support the pipe properly.

Step 2. Ring load (w_p) for 2 per cent deflection is given in Fig. 8.14 in terms of depth of cover, in feet, for soil weighing 125 lb/cu ft. The depth of cover may be converted to pounds per square inch by referring to Table 8.4. Figure 8.14 shows the height of backfill carried without ring aid by pipe with a given diameter and thickness. If the proposed pipe will carry the entire backfill load, and any live load present, no further calculations are necessary.

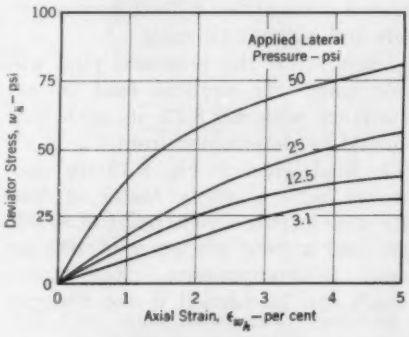

Fig. 8.19. Axial Strain in Typical Soil: Sand With Clay Binder

Data are from Test 10-T, Ref. 12.

Figure 8.14 is based upon the standard formula for an elastic ring, using a constant of 0.10, which applies to earth load on pipe in a flat-bottom trench. The constant for concentrated loads top and bottom is 0.149.

Step 3. If the ring load for the pipe almost carries the vertical loads, the engineer may elect to figure such assistance at the side of the pipe as will be given by some percentage of the active pressure of the earth in the backfill. Actually, underground pipe has probably been getting such aid for a long time without its being recognized. The value of the active pressure to be used must remain a matter of judgment based on circumstances of the installation. Unless the value is better known, the suggestion given in Paragraph 8.11b may be followed. The foregoing statements are made on the assumption that the trench is backfilled by ordinary methods and without tamping.

Step 4. If the proposed pipe will not carry the vertical load in accordance with Eq 8.10, its arch load should be determined from Fig. 8.15. The loads given in Fig. 8.15 are associated with a safety factor of four for steel having a yield point of 30,000 psi and a joint efficiency of 100 per cent in compression. Still higher loads can be carried if the designer wishes and if care is taken in the installation procedure to provide adequate strength in the earth envelope.

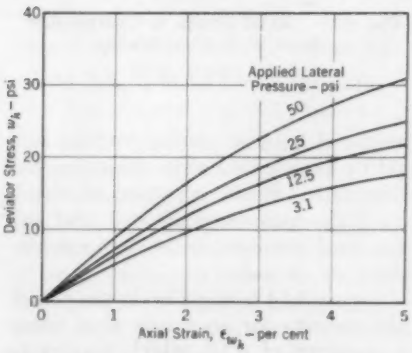

Fig. 8.20. Axial Strain in Typical Soil: Lean Clay

Data are from Test 15-T, Ref. 12.

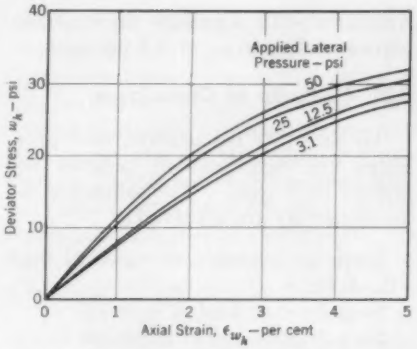

Fig. 8.21. Axial Strain in Typical Soil: Very Plastic Clay

Data are from Test 20-T, Ref. 12.

Step 5. After the effective vertical-load intensity and the ring strength of the pipe are determined, the transfer load from pipe to soil is determined using Eq 8.11.

Step 6. When the intensity of the transfer-load pressure is known, the passive deformation may be determined using Eq 8.14 or 8.15. The value ϵ or ϵ_{w_h} is a measure of the axial strain caused by an axial load on the soil while it is at the same time being subjected to lateral radial pressures due to height of cover. It is at this point that soil mechanics must enter the problem.

8.22. Triaxial Shear Tests of Soils

In Fig. 8.12, an imaginary triaxial-shear-test soil specimen is assumed to lie somewhere along the length of the equivalent earth column. If the position *C* is at the contact surface of the pipe, the specimen will be subjected

to the maximum axial stress existing in the equivalent earth column, and the confining stress on the specimen will be that due to the depth z. Then if triaxial-shear-test data, including the stress-strain characteristics of the sample corresponding to different confining pressures due to change in depth z, are available, it is possible to solve Eq 8.14 or 8.15.

Stress-strain diagrams of certain "typical" remolded soil materials compacted to 98 per cent maximum density are shown in Fig. 8.18–8.21. These diagrams give the usable axial compression in pounds per square inch corresponding to given lateral pressures in pounds per square inch, with axial strain given as percentages. The diagrams are presented as aids to understanding rather than as solutions for particular problems. Full

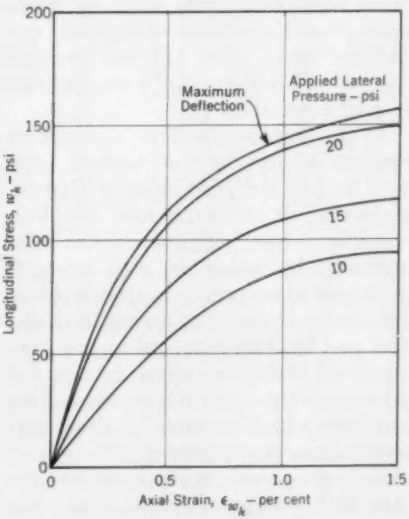

Fig. 8.22. Axial Strain in Graded Gravel at Maximum Density

Data are from Ref. 16.

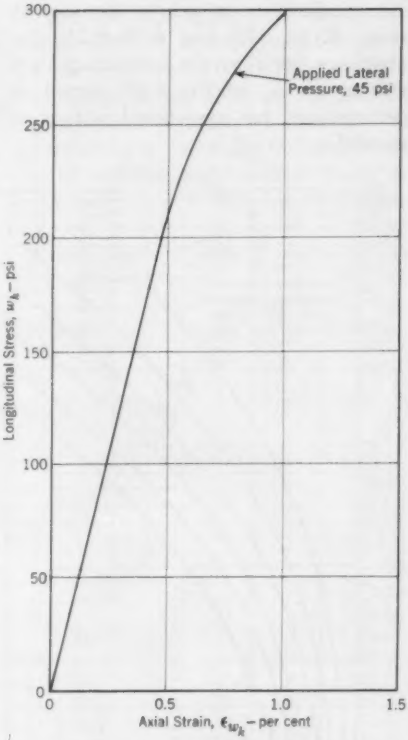

Fig. 8.23. Axial Strain in Crushed Slag at Maximum Density

Data are from Ref. 16.

data on these soils and the tests conducted by the Bureau of Reclamation have been published (*12*). Figures 8.22 and 8.23 show data for graded gravel and crushed slag for maximum compaction. Additional data have been published (*16*).

The deviator stress ordinates in Fig. 8.18–8.21 and the longitudinal stress ordinates in Fig. 8.22 and 8.23 are values of w_h found by use of Eq 8.11 and associated with the proper applied lateral pressure correspond-

ing to distance z below the surface (Fig. 8.12). In Fig 8.18–8.24, the abscissas (axial strain percentage) are values of ϵ_{w_h} in Eq 8.15 stated in percentages for associated values of w_h and z.

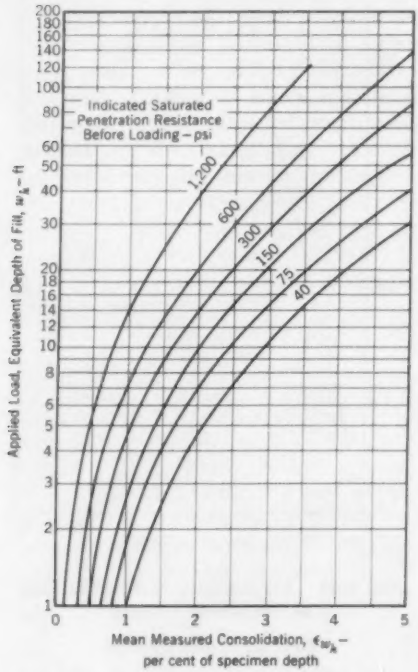

Fig. 8.24. Axial Strain or Consolidation of Soil Specimens Under Different Conditions

The various specimens were under different degrees of compaction subjected to pressures induced by applied loads or weight of cover (4, 13).

8.23. Comments on Design Factors in Barnard Theory

8.23.1. *Permissible deflection.* Deflection of unlined pipe, or of pipe lined after installation, may safely reach 5 per cent of nominal diameter. Deflection of pipelines with spun coal-tar enamel is best held to 2 per cent for mechanical field joints; however, other types of field joints may be deflected as much as 5 per cent. The deflection of pipe lined with spun cement-mortar should be held to 2 per cent.

8.23.2. *Definition of failure.* At this point, when deflection is mentioned, the engineer accustomed to thinking in terms of flexure or bending-moment formulas in rigid construction is likely to contend that permanent deflection can occur only after the yield point has been passed and that, therefore, a pipe so stressed has failed structurally and is dangerous. The simplest rebuttal to this argument is to recognize that the steel in a finished pipe has, in the manufacturing process, been cold coiled, uncoiled, bent, curved, or twisted a number of times and has been stressed beyond the yield point each time; yet, after all these operations have been completed, the finished steel pipe is used for all manner of high-pressure work without fear or hesitation.

If the engineer still is hesitant to restress a part of the finished pipe wall beyond the yield point by slightly deflecting it underground, let him consider what happens to the test specimen by which the pipe strength is measured according to specification. Usually it is sliced as a ring from the end of a finished pipe, cut at one side, uncurled from the circle into a flat piece, and then put in a tensile-testing machine which proceeds to show that after once more passing the elastic limit, the steel still possesses the specified strength. In a way, the deflection underground is simply a finished forming operation.

Therefore, where steel pipe such as is here discussed is concerned, the

word "failure" must define a state of falling short of satisfactory performance and not a state in which localized stresses appear to pass the yield point of the material as judged by the results of bending-moment formula analysis.

8.23.3. *Vertical loads.* The results of many tests show that flexible steel pipe frequently carries *less* load than that indicated by the weight of the prism directly over the pipe. Some measurements on flexible pipe have shown the full weight of the prism in flexible pipe have at times shown the horizontal intensity to be about 25 per cent more than the vertical. The shape factor used in the formula for transfer pressure intensity is intended to account for increased load due to increase in horizontal diameter, increased pressure because of shortening of vertical diameter, and increased peak horizontal intensity due to load distribution.

In the original concept of contact pressure and "sidewise settlement,"

TABLE 8.4

Unit Earth Loads Due to Height of Cover
(Soil Weight 125 lb/cu ft)

Height of Cover, *H* ft	+0	+1	+2	+3	+4	+5	+6	+7	+8	+9
	Unit Earth Load—*psi*									
0		0.87	1.74	2.60	3.47	4.34	5.20	6.08	6.94	7.81
10	8.68	9.55	10.4	11.3	12.2	13.0	13.9	14.8	15.6	16.5
20	17.4	18.2	19.1	20.0	20.8	21.7	22.6	23.4	24.3	25.2
30	26.0	26.9	27.8	28.6	29.5	30.4	31.2	32.1	33.0	33.9
40	34.7	35.6	36.5	37.3	38.2	39.1	39.9	40.8	41.7	42.5
50	43.4	44.3	45.1	46.0	46.9	47.7	48.6	49.5	50.3	51.2
60	52.1	52.9	53.8	54.7	55.6	56.4	57.3	58.2	59.0	59.9
70	60.8	61.6	62.5	63.4	64.2	65.1	66.0	66.8	67.7	68.6
80	69.4	70.3	71.2	72.0	72.9	73.8	74.6	75.5	76.4	77.3
90	78.1	79.0	79.9	80.7	81.6	82.5	83.3	84.2	85.1	85.9
100	86.8	87.7	88.5	89.4	90.3	91.1	92.0	92.9	93.7	94.6

above the pipe transmitted to the pipe. As carrying the full weight of the prism would probably be the worst case for flexible pipe, the full-weight assumption should be used. A simple empirical means of varying the load would be to modify the estimated weight of the soil prism itself. The height of cover over large pipe may be taken as the average between the horizontal diameter line and the top.

8.23.4. *Horizontal pressure.* Measurements of the horizontal pressure the value of the shape factor F was given by Barnard in an unpublished work as $F = 1.35$ for round pipe whose horizontal diameter was increased 2 per cent by vertical load. The original value of $F = 1.35$ was developed by consideration of change in pipe shape, load distribution at the extremity of the horizontal diameter, measured pressures at the side of installed pipe, and the length of the equivalent earth column from theory and as influenced by experimental observations. The value $F = 1.35$, de-

veloped through a method entirely different from instantaneous-radius considerations, is in close agreement with $F = 1.40$ in Table 8.3, developed by the latter more general method. Moreover, the value of $F = 1.35$ was used originally to check the measured deflections of many steel pipe installations of large and small diameters, thick and thin walls, under high and low cover, and with and without live load, for the purpose of proving the sidewise settlement theory sound.

TABLE 8.5

Approximate Unit Loads Due to H-20 Truck and Cooper E-72 Effective Live Loads on Structure Under Given Height of Cover

Height of Cover, H ft	Load—psi	
	H-20	E-72*
1	11	30
2	6	25
4	3	18
6	2	14
8	1	10
10	1	7
12	1	5
14		4
16		4
18		3
20		2
25		1

* Loads include 50 per cent impact.

The agreement was good between calculated and measured deflections. In the examples in Sec. 8.29, either one, or both, values of F are used as a matter of interest.

8.23.5. *Value of* ϵ. Strain, or shortening under compression, in soil varies with such factors as void ratio, moisture content, axial load, confining pressure, load repetition, and time. The "sidewise settlement" or passive deformation of the soil around the pipe likewise varies with these factors. It should be noted that, while only horizontal thrusting forces are here considered, the same general principles can apply to forces radiating from any part of the pipe periphery if the thrusting loads are known.

8.23.6. *Backfilling.* In extensive large-diameter flexible pipe installations, if vertical deflection is a serious factor, laboratory tests on backfill material are essential, unless guessing is preferred. Field compaction control should tie field operations to laboratory data unless, again, guessing is preferred. Such field control is now common practice in large highway projects and pays off in the form of better roads and lowered maintenance costs. A nominal amount of backfill control can reduce pipe material costs on large-diameter pipe installations.

One approximate method of predicting settlement is to use the Proctor (*13*) indicated saturated penetration resistance in conjunction with measuring soil dry weight, both operations being performed in the field. Percentage of consolidation can then be estimated using Fig. 8.24.

8.24. Ellipsing, Stulling, or Strutting

It is common practice in the highway and railroad field to increase the vertical diameter of round pipe during installation by installing vertical columns inside the pipe. The amount of ellipsing has been a matter of judgment and experience. The method herein proposed for determining the passive deformation at the side of the pipe provides an immediate and extremely simple method of determining if ellipsing is needed and to what extent the pipe diameter should be elongated vertically. Because the

"sidewise settlement," or increase in horizontal diameter, of the pipe is similar to the deflection of a beam or bridge under load, ellipsing is the equivalent of "cambering." When bridges are cambered, an upward extending "bow" is built into them, and the amount of camber or bow is such that, when loaded, the beam becomes straight or the bridge floor conforms to the design contour. In these cases, the natural deflection of the structure under load occurs prior to the time the deflection becomes objectionable.

TABLE 8.6

Backfill Classification According to Soil Compaction Requirements

Class	Type of Backfill	Standard ASTM Percentage of Compaction
1	Supporting extremely heavy loads	100+
2	Subject to surface loads	95–97
3	Not subject to surface loads	90–93
4	Where settlement causes no harm	85–90

From Sowers (17)

The same thinking can apply to the passive deformation of the soil at the side of the pipe. For instance, if the axial strain in the soil reaches 2.5 per cent to develop a certain transfer load, the theoretical length of the equivalent earth column can be increased 2.5 per cent by pulling the pipe side *inward* by that amount. If then, under these circumstances, the pipe is stulled or strutted 5 per cent vertically, the full transfer load will be accepted by the two equivalent earth columns, each with a 2.5 per cent passive deformation. Alternatively, if 2 per cent deflection were to be allowed in the pipe finally, 3 per cent elongation of the vertical diameter would achieve the desired result. From this it can be seen that percentage of ellipsing distance is obtainable directly through the use of Fig. 8.18–8.24. The pipe theoretically then becomes round after installation.

The stulls or struts may be removed before backfill reaches full height if it is determined that adequate side pressure has developed. If the pipe is formed and installed as an ellipse with the long axis vertical, no stulls or struts are necessary. Struts cannot be used with some types of field joints.

8.25. Trench Conditions

When pipe is put in a trench instead of a fill, that portion of the equivalent earth column lying between the side of the pipe and the side of the trench undergoes passive deformation which may be equal to that of the soil beyond the trench side only if the compaction between the pipe side and the side of the trench is such that it is comparable with the remainder of the length of the equivalent earth column.

When the backfill is not placed at 98 per cent of maximum density, the additional passive deformation in excess of that shown by the soil data figures may be estimated. Establish the distance l between pipe wall and trench side. An *extra* amount of passive deformation will occur in this portion of the equivalent earth column. This *additional* deformation is approximately 0.5, 0.7, or 1.0 times the deformation which would be calculated for the distance l for standard ASTM D698 densities of 95, 93, and 90 per cent, respectively. This additional-deformation figure is not a percentage but a distance and is to

be so considered in estimating the pipe deflection.

For example, suppose a passive deformation of 1 per cent occurs at the side of a pipe whose wall is 12 in. from the trench side when the transfer pressure is developed. If this is a 48-in. pipe, the vertical deflection is 2 per cent of the diameter, or 0.96 in. The compaction is to be 90 per cent standard ASTM density. The *additional* deflection between pipe wall and trench is $0.01 \times 12 = 0.12$ in. for one side. The total pipe deflection is 0.96 plus twice 0.12, or 1.20 in.

It has been found that, for standard ASTM D698 densities below 90 per cent, the soil resistance drops off very rapidly.

8.26. Compaction Control

The maximum density, as defined by standard ASTM D698 tests, is that obtainable with a moderate amount of rolling or tamping with the soil at optimum moisture content. The ratio of the actual soil density to the maximum density is the percentage of compaction. This indicates the degree of compaction and the ability of soil to carry heavy loads without settlement. Table 8.6 shows classification according to compaction requirements (*17*). Data on methods of trenching and backfilling for water pipe based on a soil mechanics approach have been published (*17, 18*).

8.27. Internal Pressure

Buried steel pipe when empty is under compressive forces due to soil weight and soil pressure. The outside vertical forces tend to flatten the pipe and create tensile and compression stresses in the pipe wall owing to a decrease in vertical diameter. When internal pressure is introduced into the pipe, the first action is reversed and the internal pressure tends to force the pipe to a perfect circle. The internal pressure reduces the stresses caused by the soil. In a way of speaking, the backfilling operation has prestressed the pipe against internal pressure. For the Case 3 (Sec. 8.12) design condition—full arch load and no internal pressure—the pipe wall will be under 7,500 psi compressive stress if the full cover height is used and will be under proportionally lower stress for lesser heights. Whatever compressive stress due to soil action exists in the pipe must be overcome by the tensile force due to internal pressure before the pipe wall can go into tension.

For the above reason, the common practice of determining steel pipe wall thickness on the basis of yield strength to resist internal pressure is sound and conservative. Theoretical bending stresses and tensile stress should not be totaled for steel pipe as must be done for rigid pipe.

8.28. Live Load and Shallow Cover

The equation for intensity of contact pressure (Eq 11; $F = 1.40$):

$$w_h = 1.40(w_v - w_p)$$

contains a means of determining the ring strength necessary to keep the transfer pressure to its reasonable limit. Where no live loads are present, or where the depth of cover is considerable, ring strength as such is relatively unimportant. For live loads and shallow cover, however, the quantity $(w_v - w_p)$—that is, the difference between the vertical load on the pipe and its ring strength load—must

be such that the allowable transfer load is not exceeded. An example will illustrate the procedure for determining the wall thickness in this case.

Assume: 48-in. diameter pipe, 2 per cent deflection, 3-ft cover, H-20 live load, burial in sand-clay binder as in Fig. 8.19. From Tables 8.4 and 8.5, $w_v = 2.6 + 4.5 = 7.1$ psi. From Fig. 8.19, the maximum w_h for 1 per cent passive deformation is taken as 8.0 psi. Then:

$$w_h = 1.40(w_v - w_p)$$
$$8.0 = 1.40(w_v - w_p)$$
$$8.0 = 1.40(7.1) - 1.40w_p$$
$$8.0 - 9.94 = -1.40w_p$$
$$w_p = 1.4 \text{ psi}$$

From Table 8.4, 1.4 psi corresponds to a 1.6-ft cover. From Fig. 8.14, for a 1.6-ft cover, the thickness of a 48-in. pipe should be 0.375 in. under the live load in this soil compacted to 98 per cent density. If there were no live load, Fig. 8.15 shows that this pipe could be as light as 0.141 in. and still not deflect 2 per cent.

8.29. Examples of Calculation

Problem 1 (self-supporting conduit —Case 1, Fig. 8.13a—where *no side support* is assumed to be provided by the soil). What is the height of cover without live load for a 30-in. pipe having $\frac{1}{2}$-in. wall thickness when allowing a maximum deflection of 2 per cent?

Solution. Referring to Fig. 8.14, the height of cover is found to be 21 ft.

Comment. Backfilling may be done with no special attention to tamping because the pipe carries the full load by ring strength.

Problem 2 (testing for semirigidity —Case 2, Fig. 8.13b). Will a 30-in. pipe having a $\frac{1}{4}$-in. wall thickness sustain a 5-ft cover plus an H-20 live load without exceeding 2 per cent deflection? Backfill is sandy clay.

Solution:

Step 1. Vertical-load intensity equals 4.34-psi dead load (Table 8.4) plus 2.5 psi (Table 8.5), totaling 6.84 psi.

Step 2. From Fig. 8.14, a dead load of 2.7 ft of cover will be carried by ring strength. A cover of 2.7 ft is equivalent to 2.34 psi ($\frac{1}{10}$ of 23.4 psi for 27-ft cover; Table 8.4).

Step 3. Is this a Case 2 situation? Assume ring aid (active soil pressure) equals $\frac{1}{3}$ of vertical soil pressure at depth of pipe axis (6.25 ft). Pressure at depth of 6.25 ft (this is height z, Fig. 8.12) equals 5.4 psi (interpolated from Table 8.4). Assumed active pressure is $\frac{1}{3}$ of 5.4 = 1.8 psi. Referring to Eq 8.10:

$$w_v - w_a = 6.84 - 1.8 = 5.03$$
$$5.03 > 2.34$$

As 5.03 is greater than 2.34, this cannot be considered Case 2 and must be calculated as Case 3—that is, $w_p = 0$.

Step 4. From Fig. 8.15, arch load is 140 ft, which is more than adequate.

Step 5. With $F = 1.35$ in Eq 8.11,

$$w_h = 1.35(w_{vd} + w_{vl})$$
$$= 1.35(4.34 + 2.5)$$
$$= 9.23 \text{ psi}$$

If $F = 1.40$ is used instead of $F = 1.35$, $w_h = 9.57$ psi. The difference of 0.34 psi is too small to detect in Fig. 8.19.

Step 6.

From Fig. 8.19, it is found (by interpolation between Curves 3.1 and

12.5) that for a z height corresponding to 5.4 psi (see Step 3) the axial strain, ϵ_{w_h}, for $w_h = 9.2$ psi is about 0.8 per cent. From Eq 8.15,

$$\Delta L = \epsilon_{w_h} D$$
$$\Delta L = 0.008\ (30) = 0.24 \text{ in.}$$

As this is less than the 0.01 (30) = 0.30 in. allowable, pipe will be satisfactory provided backfill is similar to that represented by Fig. 8.19 and is tamped to about 98 per cent of optimum density.

Comment. In the foregoing example, notice that there is a leeway of 25 per cent between the 0.24-in. theoretical deflection and the 0.30 in. allowable. Experience indicates this to be an ample lag factor, or time factor, to account for final soil consolidation and passive deformation.

Problem 3 (flexible conduit). A 30-in. smooth-wall pipe with 0.109-in. wall thickness is to be installed with tamped sand backfill. What vertical deflection is to be expected under covers of 4, 8, and 12 ft, assuming that Fig. 8.18 is typical of the backfill material—graded sand?

Solution:

Step 1. From Table 8.4, $w_{vd} = 3.47$, 6.94, and 10.4 psi for 4-, 8-, and 12-ft cover, respectively.

Step 2. Ring load from Fig. 8.14 is less than 1 ft (ignore).

Step 3. No ring load, no ring aid (ignore).

Step 4. Arch load from Fig. 8.15, by extrapolation, is 60 ft and is therefore satisfactory.

Step 5. Transfer load, $w_h = 1.35 w_{vd}$ (with $F = 1.35$ in Eq 8.11). Then:

$$w_h = 4.68 \text{ for } H = 4$$
$$= 9.37 \text{ for } H = 8$$
$$= 14.0 \text{ for } H = 12$$

If $F = 1.40$ is used instead of $F = 1.35$, the 3.8 per cent increase in w_h is imperceptible in Fig. 8.18.

Step 6. Passive deformation (from Fig. 8.18):

	$H=4$	$H=8$	$H=12$
w_h—*psi*	4.7	9.4	14.0
z—*psi*	4.5	8.0	11.5
ϵ_{w_h}—*per cent*	0.3	0.6	0.7
Pipe deflection			
Calculated—*per cent*	0.6	0.12	0.14
Calculated—*in.*	0.18	0.36	0.42
Measured—*in.*	0.13	0.22	0.32

Comment. Problem 3 is a duplicate, as far as is known from reports, of the Chapel Hill, N.C., pipe test (*6*). Considering the multiplicity of factors influencing the precise installation of such a flimsy pipe, plus the assumption of properties of sand, the above agreement between estimated and actual figures appears quite satisfactory for practical purposes.

Problem 4 (large pipe, high fill). What deflection can be expected in an 84-in.-diameter corrugated pipe with a wall thickness of 0.281 in. installed with 137 ft of cover in a 100 per cent compacted granular fill having properties of graded gravel in Fig. 8.22? Pipe is to be strutted 3 per cent vertically during installation.

Solution:

Step 1. From Table 8.4, $w_{vd} = 118.9$ psi.

Step 2. Ring load—by comparison of section modulus values, this pipe has a ring load capacity about equal to 1-in. plate pipe of the same diameter. Then, from Fig. 8.14, ring load cover equals 8 ft, equivalent to 6.94 psi.

Step 3. Omitted, ring aid ignored.

Step 4. From Fig. 8.15, the arch load indicated is over 200 ft.

Step 5. Because of the very high load on this pipe, and because the final transfer pressure will depend on the final shape of the pipe, a trial calculation should be made to estimate this final shape. The deflection shown by the trial calculation will permit a closer value of F to be selected for a subsequent calculation for transfer load and deflection.

Trial calculation:

For 3 per cent strutting vertically, $F = 1.04$ (Table 8.3) in Eq 8.11:

$$w_h = 1.04(118.9 - 6.94)$$
$$w_h = 116.4$$

Step 6. Figure 8.22 does not contain a value for z distance as great as involved here. But if the curve showing laboratory maximum measured deflection is read, $\epsilon_{w_h} = 0.6$ per cent, and pipe deflection is twice this, or 1.2 per cent. The pipe with load on will be deflected 3 per cent minus 1.2 per cent, or 1.8 per cent. By interpolation in Table 8.3 between $F = 1.14$ for 1.5 per cent deflection and $F = 1.11$ for 2 per cent deflection, $F = 1.122$ for 1.8 per cent deflection.

Final calculation:

$$w_h = 1.122(118.9 - 6.94)$$
$$w_h = 136.6$$
$$\epsilon_{w_h} = 0.8 \text{ per cent (Fig. 8.22).}$$

The calculated pipe deflection is therefore 1.6 per cent or 1.34 in.

Comment. Problem 4 postulates the conditions of the multiplate pipe installed and observed at Cullman, Ala. (*19*). Two years after completion, the maximum deflection measured at the high fill was about 1.25 in. Therefore, the calculated deflection checks the measured deflection for all practical purposes and is on the conservative side.

Problem 5 (large pipe, low cover). A 96-in.-diameter water pipe carrying 150 psi pressure is to be put in a trench under 22 ft of cover with no live load, allowing 2 per cent deflection. What thickness can be used with sandy-clay backfill?

Solution. Wall thickness required for internal pressure is $\frac{1}{2}$ in., allowing a tensile stress of 15,000 psi (Table 6.1, Chap. 6). An old rule of thumb for minimum wall thickness to meet the practical requirements of fabricating, transporting, and installing steel pipe is that this minimum should be not less than 0.5 per cent of the diameter. For 96-in. pipe, the minimum thickness would be 0.48 in. As a thickness of $\frac{1}{2}$ in. also satisfies the internal-pressure requirement, further calculations will be based on the use of $\frac{1}{2}$-in. wall:

Step 1. From Table 8.4, $w_{vd} = 19.1$.

Step 2. From Fig. 8.14, ring load is less than 1 ft (ignore).

Step 3. Omitted.

Step 4. From Fig. 8.15, arch load is 90 ft. As cover is only 22 ft, $\frac{1}{2}$-in. wall thickness is adequate.

Step 5. From Eq 8.11, transfer load $w_h = 1.40(19.1) = 26.7$ psi.

Step 6. From Fig. 8.19, for $z = 26$ ft or 22.6 psi, $\epsilon_{w_h} = 1.7$ per cent; pipe deflection equals $2\ (1.7) = 3.4$ per cent.

Therefore, this pipe can be held to 2 per cent deflection with fully compacted backfill of sand with clay binder, provided it is strutted $3.4 - 2.0 = 1.4$ per cent (about $1\frac{5}{16}$ in.).

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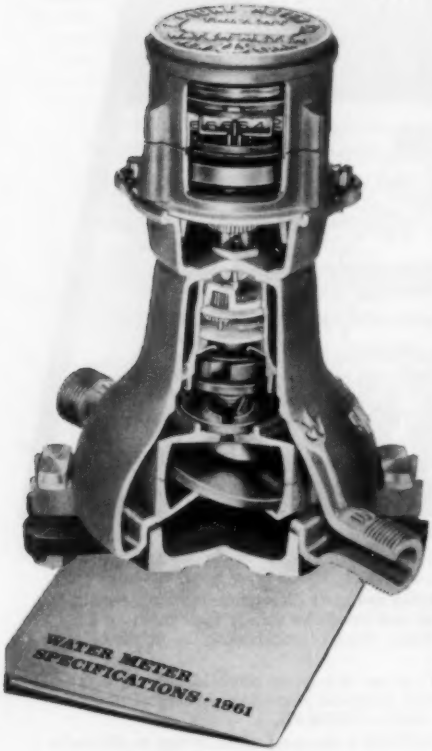

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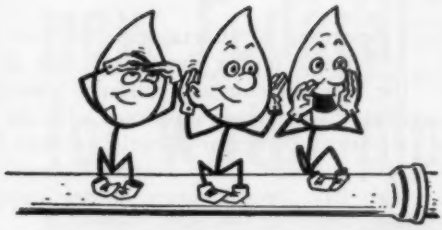

Percolation and Runoff

'There is no water shortage; there is only a shortage of the facilities to collect, store, treat, and deliver water to your tap.'

That was the point that Caesar Wertz and the other officers of AWWA were plugging hard at the Detroit Conference in June. That was a point that Abel Wolman also bore down upon in a mid-May interview which appeared in the *American Weekly* magazine, noting:

> There is no water crisis. But there is a water panic. . . . There is no water problem in this country now or in the foreseeable future which cannot be solved by more scientific management of our current local water supplies or by importation of water from other areas.

That is a point which is going to take a lot of making, however, in the face of headlined "shortages" across the country. After a virtually snowless winter, headlines in Utah were full of the disastrous drought, but the Salt Lake *Tribune* editorial page was able to point out: "Thanks to Deer Creek storage, Salt Lake City should have an adequate supply without restrictions." At Marion, Ind., a city of 38,000, headlines told of "an acute water shortage" that closed down schools and industry and put stringent restrictions on home uses, caused by a sudden dropping of water level in its well field. But more water was at hand as quickly as two new wells could be drilled in a different field. And in Northern California, generally referred to as an area of surplus water, there were headlines about a "critical water shortage" as the temperatures in the San Francisco area hit the hundred mark, but the San Francisco water department was able to take the 50 per cent jump in consumption in stride and talk of the $115,000,000 expansion blueprinted to keep ahead of future such emergencies. Unfortunately, there seems to be much more news in undergoing a crisis than in averting one and water utility men themselves often have trouble seeing beyond their own intakes.

But the problem is more than one of myopia; there is acute astigmatism as well, resulting in a picture that is sometimes blurred, but more often completely out of focus. A basic cause of the distortion is the lack of distinction between "use"—the amount withdrawn from a source of supply for any purpose—and "consumption"—the amount which is changed in form by its use (through evaporation, trans-

(Continued on page 36 P&R)

(Continued from page 35 P&R)

piration, and other processes) so that it is no longer available as water. It was such a failure to distinguish between consumptive and nonconsumptive uses—together, no doubt, with a familiarity with deficit spending—that led the House Committee on Science and Astronautics recently to announce that the nation's consumption of water was already greater than its available supply. And it was certainly a similar failure that led Assistant Secretary of Interior Kenneth Holum to point out to the Missouri Basin Inter-Agency Committee on May 25 that:

> Water is becoming a more precious asset every day we live. In every area of the nation the demand for clean, fresh water threatens to outrun the available supply.
>
> Today Americans are consuming 60 per cent of our fresh water supply. By 1975, with our tremendous growth in population, we will be consuming 90 per cent of the supply presently available. The USGS predicts that by 1980 we will be using 600 bgd. This is double the present rate [twice 60 per cent, that is].

although less than 10 days later USGS engineer Kenneth MacKichan was reporting to AWWA in Detroit that present consumption totaled only 61 bgd, compared to an available supply estimated at 600 bgd.

Out of such confusion, the "panic" of which Abel Wolman speaks, rather than the necessary public understanding, will come. But before the public can be taught, water works men themselves will have to appreciate the fact that water is not short, it is only distant or difficult or expensive to obtain.

There is no water shortage, there is only a shortage of the understanding and action required to make it available when and where it is needed!

Speaking of shortages is not only a domestic matter these days, nor is the cure therefor any different whatever the language involved. In the Americas, particularly, stories of shortages and activity to overcome shortages have been frequent in the past three months.

From Brazil, last March, came word that Rio de Janeiro was virtually waterless after a flash flood damaged the water pumps. With most of the city's 3,500,000 population completely without water, some for as long as 2 weeks, and with temperatures reaching the 90's, even the school children, forced by the emergency to stay home, were beginning to look forward to a bath. And in the state of public "calamity" proclaimed by the governor, heads were beginning to roll as day after day went by without apparent progress. Long resigned to disruptions of service because of "an antiquated and collapsing storage and distribution system," Rioans were beginning to thirst for more than water.

From Cuba in April came word of another "virtually waterless" metropolis as saboteurs blew up water mains in Havana "in defiance of the warning from Premier Fidel Castro that there may be shortages in Cuba but there are no shortages of bullets for counter-revolutionaries."

From Ecuador, the story was one of a $100 CARE water pump, given to the town of Riombabas in the Andes, changing the life of a community that had been scourged by an impure and inadequate water supply.

From Nicaragua, it was reported that President Somoza and US Ambassador Thomas E. Whelan had joined forces to dedicate a new $600,000 water plant at Metagalpa,

(Continued on page 38 P&R)

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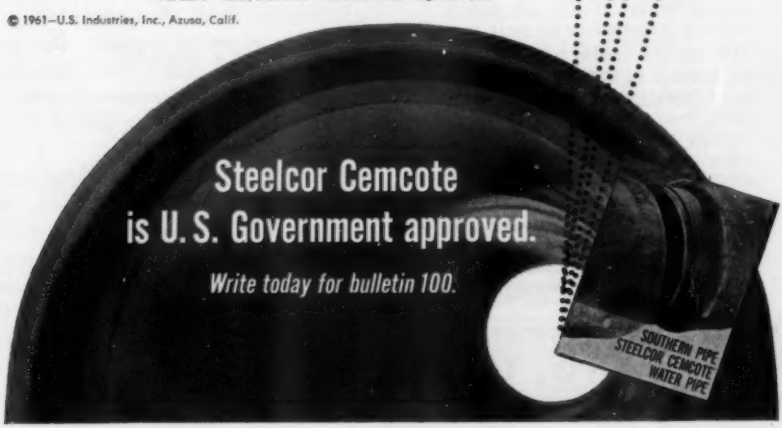

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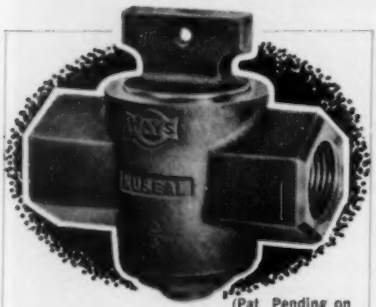

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(*Continued from page 36 P&R*)

built with funds borrowed from the Inter-American Development Bank.

From Peru, the news is that a $3,900,000 loan has been made to improve and expand the water and sewerage system in Arequipa, also from the Inter-American Development Bank.

And from Venezuela, the shortage story stems from the driest spring in 43 years, putting Caracas' 1,200,000 population on short rations and dispatching air force planes with dry ice to seed the clouds in the hope of breaking the drought.

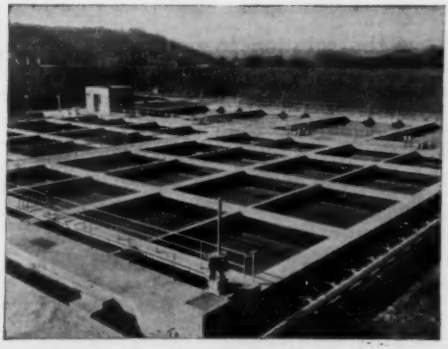
Not to see America thirst, President Kennedy's Ten Year Plan for the Americas has placed water supply second only to land reform in funds allotted from the first installment of the Inter-American Fund for Social Progress.

Your customers may start laughing one of these days. If they do, you can figure that Captain Easy has not yet caught up with Alonzo Ameeba, inventor of a happiness serum "so highly concentrated that an iota could set a thousand people slapping their thighs." When last seen Alonzo was contaminating the city's (and we're not sure which) drinking water supply with "smiles, grins, and chuckles," not to mention a guffaw or two. But perhaps you don't read the "funnies." For sure, Leslie Turner, creator of Captain Easy, reads the JOURNAL.

That water can level its own seekers was proved in Ipswich, Mass., last June, when town fathers decided that the best way to make delinquent water customers pay their bills was to shut off their water supplies when the temperature climbed over 90°F. The money flowed in.

(*Continued on page 40 P&R*)

WATER LEAKAGE PROBLEMS IN CONCRETE STRUCTURES?



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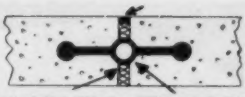

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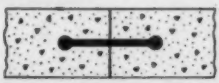

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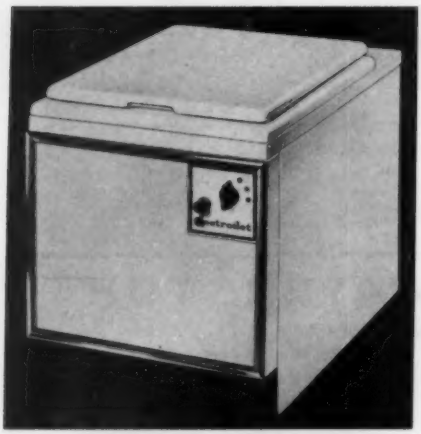
(Continued from page 38 P&R)

That the solution to pollution is not dilution but combustion is what LaMere Industries, Inc., of Walworth, Wis., seeks to prove with a squat squat box (see cut) called the Destroilet. Both waterless and nonchemical, this unplumbed john gets rid of human wastes by burning them with a gas flame at extremely high temperature, leaving so little ash that a vacuuming every 6 months is all that is necessary for cleaning. Advantages claimed for the new hot seat are:



1. Water, septic systems, sewage pipe, etc., are no longer needed for human waste disposal. [But then most of us are inhumanly wasteful, too.]

2. Destroilet may be used anywhere . . . forget the limitations of conventional toilets. [We do have an electric connection on our terrace, but no gas pipe.]

3. As opposed to conventional toilets, Destroilet is always ready for use without time intervals for water to reach required levels. [When we gotta go, we can usually wait.]

4. Built-in exhaust fan sucks out 100 per cent odor . . . result: for the first time, an odorless bathroom. [Correction: an odorless john, our bathroom has an exhaust fan.]

5. Destroilet embodies economy. Avererage cost per individual, only pennies per day; savings per individual, approximately 25 gallons of water per day. [Even making us a fourflusher doesn't add up to money saving there.]

Even so, we will admit that the Destroilet has a place—as an inhouse we mean.

The assault on salt not only continues, but expands and accelerates, with President Kennedy himself taking a direct interest in enlarging the present program of the Office of Saline Water. Thus, at the end of June, following the official opening of the 1-mgd Freeport, Tex., pilot plant, he sent to Congress a message suggesting legislation to increase the time limit on the current research program, to remove the financial ceiling on research, to provide for additional pilot plants of capacities up to 50 mgd, and to offer federal aid to communities to encourage their construction of conversion plants for municipal supply. Meanwhile, *Chemical Week* magazine, in a review of the current industrial surge into desalting, estimated that the prize for a practical and economical system was $100 billion in construction contracts in the next 20 years.

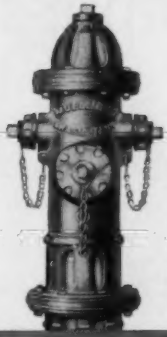
Indicating an interest in that kind of sugar, in addition to the ten companies listed in July P&R (p. 38), were:

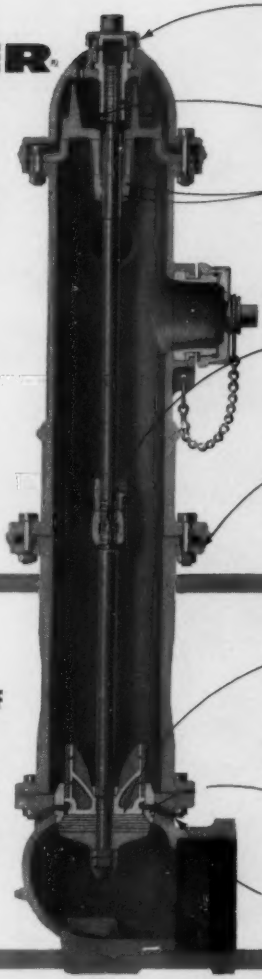
Aqua-Chem, Inc. (formerly Cleaver Books Special Products), a pioneer in the manufacture of flash evaporation and vapor compression equipment, that has now taken over the distribution of a new electrodialysis process devel-

(Continued on page 44 P&R)

MUELLER

AWWA IMPROVED FIRE HYDRANTS





Bronze Weather Cap—Keeps water away from operating nut; prevents tampering.

Oil Reservoir—Automatically lubricates stem threads, bearing surfaces.

"O" Ring Seals—Threads and bearing surfaces are sealed from water by permanent, watertight, adjustment-free seals.

Safety Stem Coupling—Breaks apart without bending the stem. Repaired in minutes without digging and with no loss of water.

Safety Flange—Separates cleanly on impact. Prevents damage to barrel. Restored without shutting off water.

Compression-type Main Valve—Closed by water pressure. Of special wear and damage-resistant material. Permits repairs without shutting off water.

Bronze Seat Ring—Straight threads, with copper-asbestos gasket, permits easy removal.

Double Drain Valves—Empty barrel completely. Flushed with use.

WEATHERPROOF
TAMPERPROOF
MAINTENANCE-FREE!

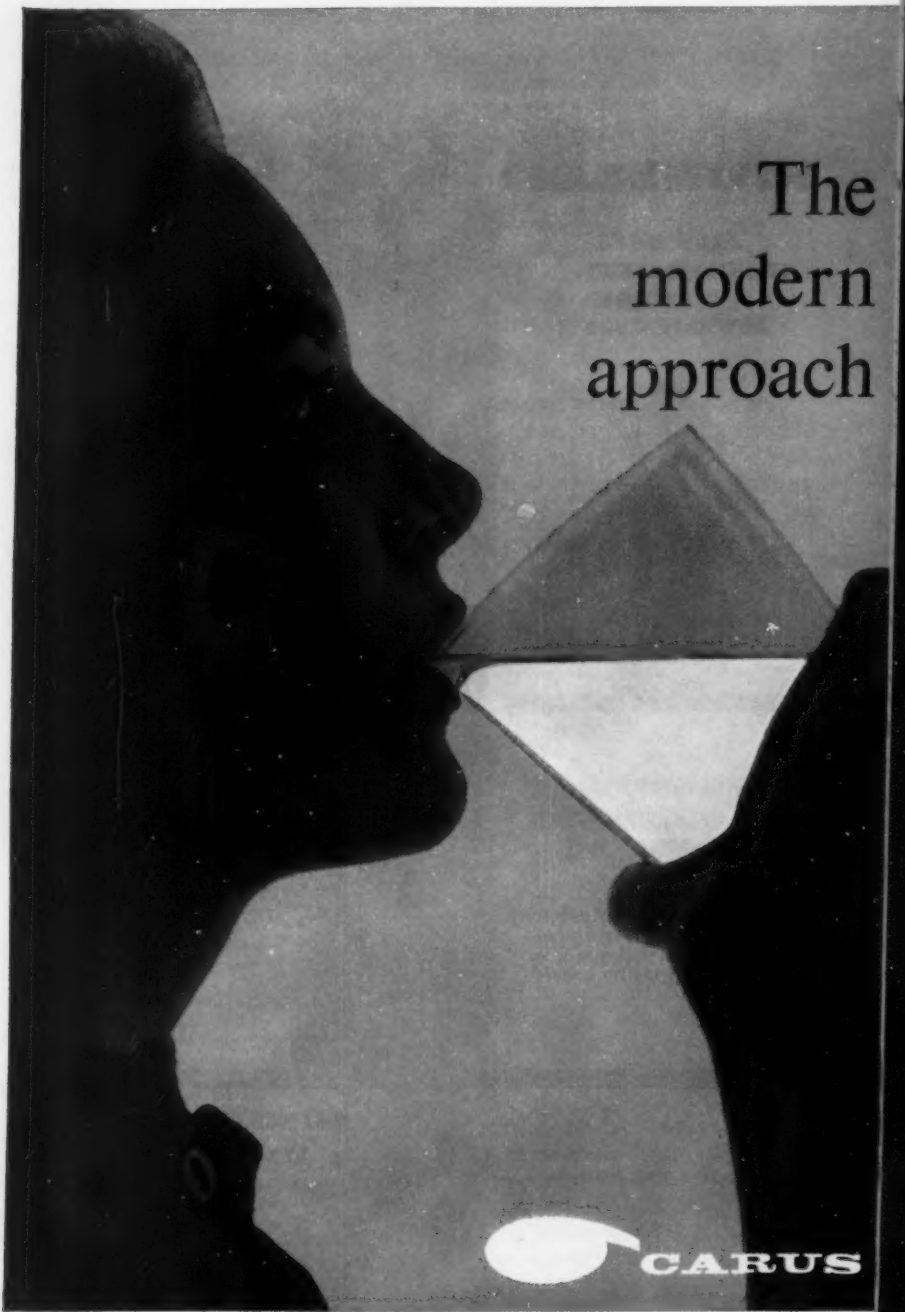
Once set, you can forget about a Mueller Improved Hydrant—the Mueller Hydrant is always ready for action.

Write for information and specifications.

MUELLER CO.
DECATUR, ILL.

Factories at: Decatur, Chattanooga, Los Angeles
In Canada: Mueller, Limited, Sarnia, Ontario

06371


The
modern
approach
CARUS

to taste and odor control

CAROX* Potassium Permanganate

Does the Job . . . and Does it Better . . .
In Large or Small Water Systems

You can provide *quality* water (and probably save money doing it!) with CAROX Potassium Permanganate. This extremely effective chemical removes obnoxious unpleasant tastes and odors caused by industrial wastes, decayed vegetation, and algae. It works quickly and efficiently, via oxidation and adsorption, to help provide safe, pure, palatable water.

You don't need any special equipment to use CAROX for water treatment; it works in virtually all conventional water treatment systems. The Potassium Permanganate method generally costs less to use than other prevalent techniques for eliminating tastes and odors; and, since it aids coagulation and serves as an effective biocide, it minimizes quantity requirements on other chemicals. CAROX is easy to handle, non-corrosive, and requires minimum storage space.

If you have taste and odor problems, you owe it to your community to investigate CAROX. Write to Carus Chemical Company, Inc., 1379 Eighth Street, LaSalle, Illinois, or phone 223-1500 for complete information.

*Trade Mark

CAROX **removes** ***musty, earthy, woody, moldy, swampy, grassy, fishy tastes and odors***

CAROX **destroys** ***hydrogen sulphide, phenols, chlorophenols, acrylates, mercaptans, organic herbicides, fungicides, insecticides***

CHEMICAL COMPANY, INC.

(Continued from page 40 P&R)

oped in the Netherlands for converting brackish water. Conversion costs for the new process in a 2.8-mgd plan in Africa were reported as 54 cents per 1,000 gal.

Clark Brothers Co. (division of Dresser Industries), which announced the availability of a new freezing process, developed by Shiji Umano of the Japanese Chemical Research Inst., reputedly capable of converting sea water at a proved cost of 35–45 cents per 1,000 gal.

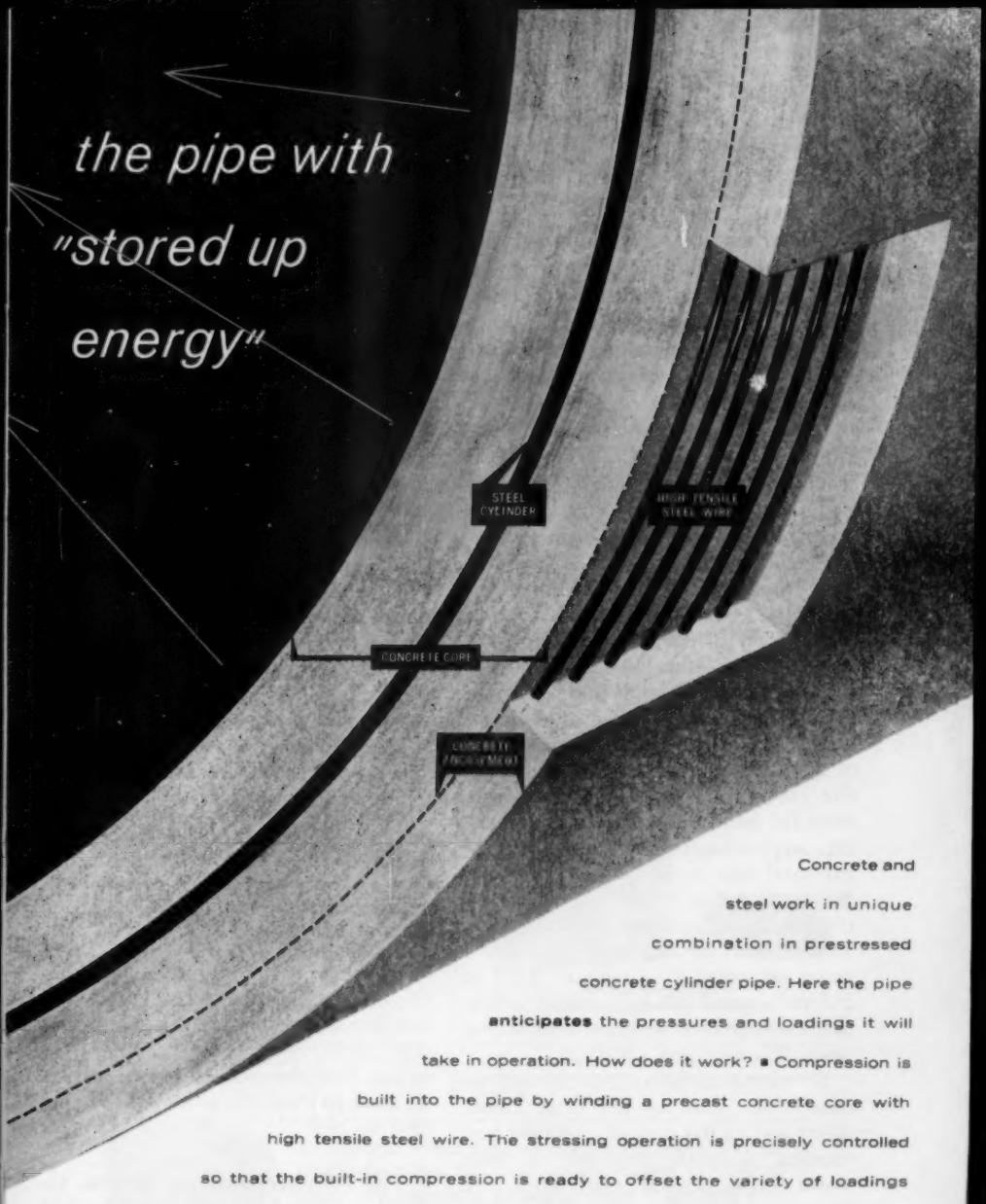
Koppers Co., which is studying the possibility of conversion by a hydrate process that will be still less costly than freezing.

North American Aviation Corp., which has estimated conversion costs using its "rocketdyne" process in a 10-mgd plant at 40 cents per 1,000 gal.

Encouraging and exciting as this progress is, we can't help dreaming about what the water utility field could do with just half that $100 billion in providing the facilities to improve the quality of the fresh water now available and to deliver it at optimum pressure to those who need it, when and where they need it. For the fact remains that the desalters still have a long, hard way to go to meet the production costs of treated fresh water. It is true that present estimates of desalting costs run

(Continued on page 46 P&R)

David B. Lee (left), director, Bureau of Sanitary Engineering, Florida State Board of Health, presented the 29th annual awards for the best operated water treatment plants in the state to the following: Walter Peele, superintendent, Fiveash Water Plant, Fort Lauderdale; Jerry Sellers, superintendent, Cocoa Water Plant; John Seller, superintendent, Vero Beach Water Plant; and Fred Kingery, superintendent, Pinellas County Water System.



Concrete and steel work in unique combination in prestressed concrete cylinder pipe. Here the pipe **anticipates** the pressures and loadings it will take in operation. How does it work? ■ Compression is built into the pipe by winding a precast concrete core with high tensile steel wire. The stressing operation is precisely controlled so that the built-in compression is ready to offset the variety of loadings the pipe will take under operating conditions. ■ The performance of this pipe is so accurately predictable that concrete and steel are used to greatest advantage. ■ Our customers have found that prestressed concrete cylinder pipe is economical for larger diameters, high pressures, or heavy loadings. ■ This may be the pipe for your next pipeline project. Let us talk it over with you.

American Pipe and Construction Co. • Los Angeles • San Diego • Hayward • Portland
Bogota, Colombia ■ **American Concrete Pipe Co. (subsidiary)** • Phoenix • Albuquerque
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A MEMBER OF THE AMERICAN CONCRETE PRESSURE PIPE ASSOCIATION

AMERICAN
PIPE AND CONSTRUCTION CO.

(Continued from page 44 P&R)

as low as 35 cents per 1,000 gal and it is true that these figures are compared with fresh-water costs of 20–70 cents per 1,000 gal, but the comparison is, to be kind, contrived. The 35 cents per 1,000 gal for desalting, we understand, is the cost of plant operation without depreciation of equipment or any of the other costs involved in getting it to the consumer. The 20–70 cents per 1,000 gal, on the other hand, is the actual selling price of domestic water supply at the highest rate. Getting a little closer to comparative figures, we might take the actual average cost of water as reported in AWWA's 1955 survey of operating data for 497 water utilities—that is, 12.3 cents per 1,000 gal. But this includes not only the cost of treatment, but that of transmission and distribution, depreciation of plant equipment, and administrative costs. The simple cost of treatment for fresh water, including softening, approximates 5 cents per 1,000 gal—and this is really the target for desalters on the municipal water supply front. In industry, the bullseye is smaller, with the actual selling price of water estimated by *Chemical Week* at 2–5 cents per 1,000 gal; and in irrigation use, almost a pinhead, with selling price at 1 cent per 1,000 gal.

Of course the assault on salt will be successful. And as the price comes down and as the cost of obtaining and treating adequate supplies of fresh water rises, desalting processes will become more and more an alternative means of obtaining fresh water—not only from the sea and from brackish supplies, but perhaps from polluted fresh ones. Meanwhile though, unless water works men think they can get away with a "cash and carry" system, they had better concentrate their efforts on inducing their customers to support them in improving their present mundane water service.

Martin F. Tiernan, chairman of the board of Wallace & Tiernan, Inc., was awarded the Msgr. John L. McNulty Memorial Medal by the College of Medicine and Dentistry of Seton Hall, South Orange, N.J., for his services to the fields of medical and dental education.

Oliver Barklage is unfair to American womanhood! That was the conclusion drawn by most readers of the New York *Daily News* last May, when Oliver, on a visit to New York from Omaha, stopped long enough to give "The Inquiring Fotographer" this statement on the subject of glamor:

> Certainly in my field glamor would not promote or improve efficiency. I'm also inclined to believe that too much glamor in any office would decrease efficiency materially.

At this point we have to admit that his "field" is our "field" and that Oliver is a member of AWWA—in fact has been one for 20 years—and that, furthermore, he is a peddler. So owing him the benefit of the doubt, we have to think his wife was standing next to him.

The softener-hardener device, for which we called a year ago (July 1960 P&R, p. 48) upon the announcement of a link between soft water and heart disease, had better be invented soon, for a British research team has just confirmed the findings of the Dartmouth doctor who first found that

(Continued on page 48 P&R)

Protective Coating Inspection Problems?

Here's Your Answer

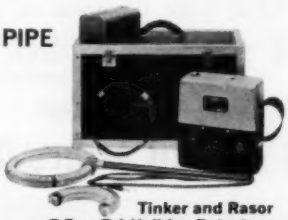
No matter whether you're coating a pipeline or a water tank, continuity is vital. Use of a Tinker and Rasor Holiday Detector while the job is open, can save days of downtime later on.

• THIN FILM

Tinker and Rasor
M-1 Holiday Detector

For painted or sprayed thin film coatings such as vinyls and epoxies. Maximum applied voltage 67½ V., non-destructive to coatings. Belt mounted, 4-ibs. total weight.

• PIPE



Tinker and Rasor
E-P or E-4 Holiday Detectors

Output adjustable from 5,000 to 20,000 pulsating voltage.

E-P — All purpose for larger diameter pipe, damp or dry climate, pre-fab film or hot applied coatings.

E-4 — Lower cost, dry surface type of detector specifically designed for smaller diameter pipe and flat surfaces.

• PLANT AND YARDS

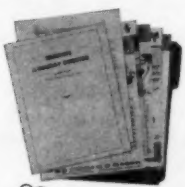
Tinker and Rasor
EPAC Holiday Detector

EPAC operates off 110 volts A.C. power for stationary coating operations. Internal voltage adjustment from 5,000 to 20,000 volts or with external variable transformer from 500 to 6,000 volts, or 5 KV to 20 KV.

• UNDERGROUND

Tinker and Rasor
Pearson-type Holiday Detector

For detecting holidays and electrical shorts without uncovering the pipeline. Completely transistorized . . . generates 15 watt, 750 cycle, stable A.C. Audio-frequency signal. Adaptable to null search method.



Tinker and Rasor has prepared a complete data kit which describes the null search system as well as other recommended procedures for inspecting protective coatings. Material includes technical data on equipment, general discussion of types of detectors, theory of operations, etc.

Quality Control for Coating Application

TINKER & RASOR

417 Agostino Road, P.O. Box 281 • San Gabriel, California

(Continued from page 46 P&R)

deaths from heart disease were more frequent in soft-water regions than in hard-water regions. To date it has not been indicated that soft water causes heart disease nor that hard water protects against it, but we have a good idea that a hard-water service could, with some imaginative advertising, be made quite profitable in the hard-artery areas. Meanwhile, in the hard-water regions, soft water service can continue profitable for other than drinking waters. As for the operators of municipal softening plants, they had better hope that the missing link is not hydraulic. As a matter of fact, we think we have noticed that people in soft-water regions prefer Scotch, whereas those in hard-water regions invariably drink bourbon!

(Continued on page 50 P&R)

This Month Years Ago

August 1861—Testing for iron in water involved "a solution of prussiate of potash. If the least iron is present, its action is instantaneous and most remarkable. Although the water charged with iron may be almost colorless, and that containing three or four drops of the test solution equally so, they will instantly form, when mixed, an exceedingly dark solution of Prussian blue. If gallic acid were used instead of prussiate of potash, the resulting mixture would be black ink."

A publication announced "the final success of the artesian well which has been sunk at Passy near Paris. The bed of water was met at a depth of 577 meters (1893 ft)." . . .

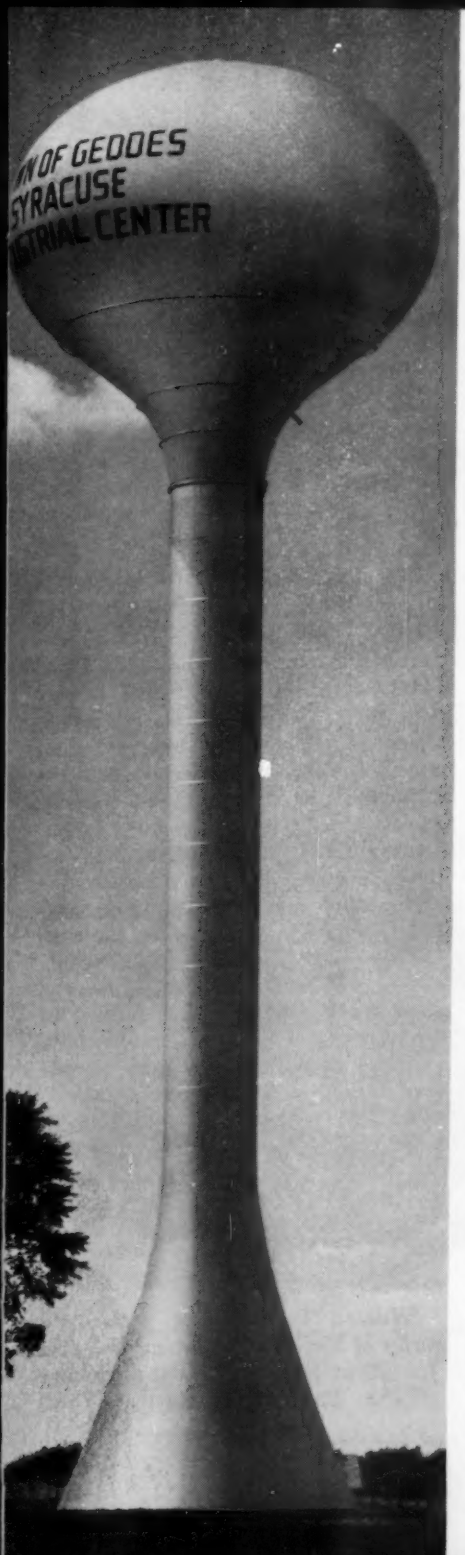
August 1911—A shortage in the water supply of a number of villages on the lower Hudson, not far from New York City, was reported. The villages included North Tarrytown, Dobbs Ferry, Ardsley, Hastings, and Scarsdale, supplied by the Consolidated Water Co.

The exhaustion of the water supply at Dallas, Tex., and Charlotte, N.C., together with the shortage at Baltimore, Md., all emphasized the necessity of providing for more adequate storage or of supplies from additional storage where communities depend upon streams of limited dry-weather capacity.

A large water filtration plant for New York City had been under consideration for a number of years, and an appropriation between $8,000,000 and $9,000,000 for construction work was made. The general proposition was to build a plant with a capacity of 400 mgd on the site of what was originally intended to be the eastern basin of the Jerome Park Reservoir.

A notable installation of electrically driven turbine pumps has recently been made at the new Diamond pumping plant of the People's Water Co., Oakland, Calif. The present installation provides for a pumping capacity of 6 mgd under a variable head of from 220 to 280 ft.

"Until recent years few attempts to restore the carrying capacity of tuberculated water mains have been made in this country, although the practice has long been common enough in England." . . . An article on pipe-scraping operations at Southmolton, England, showed that in some places it was considered worthwhile to scrape small (5-in. and 6-in.) mains.



why

Geddes, N. Y.

went

14 STORIES HIGH

to assure

better water service

for new industries

PDM
ELEVATED STEEL TANKS

PEDESTAL SPHEROID ELEVATED STEEL TANKS

This 147-foot PDM Pedestal Spheroid—world's highest tank of its type, so far as is known—brings a 500,000-gallon water capacity into balance with Geddes' expanding supply system. Serving the Syracuse industrial area with assurance of ample water facilities for new industries a-coming, the new PDM unit ties in with a 100,000-gallon tank 3 miles away for effective pressure equalization. Is *your* community preparing for increasing water demands?

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SANTA CLARA, CALIF.	P. O. Box 329
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NEWARK (2)	744 Broad Street
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ATLANTA (5)	361 East Paces Ferry Rd., N. E.
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SEATTLE (1)	500 Wall Street

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BALTIMORE
BIRMINGHAM
DES MOINES
PROVO, UTAH
CASPER, WYO.
SANTA CLARA
FRESNO
STOCKTON, CALIF.

(Continued from page 48 P&R)

Water meters attracted much attention (see cut below) at the International Agricultural Exhibition, held in Cairo, Egypt, last spring. The purpose of the demonstrations was to show how the use of metering equipment could aid irrigation programs. The meters that were exhibited were manufactured by Measure-Rite, Inc.,

of Covina, Calif., a subsidiary of Badger Meter Mfg. Co., Milwaukee.

Moses condemned engineering jargon—Robert Moses, that is, once head of parks and almost everything else in New York City and now boss of the 1964 World's Fair. This latter-day Moses was speaking at commencement exercises of the New York Law School when he put "most lawyers and almost all engineers" in a group "launched without ability to express themselves." "I would ask them," he said, and we have to agree as far as lawyers are concerned, "to devote their time to the elimination of the guff, abracadabra, and pompous lingo in which almost every profession, and notably law, clothes its writings and pronouncements to confuse the issues, impress the ignorant, and line the pockets of the printers." If the sackcloth fits, . . . !

The Fifth Congress of the International Water Supply Assn., held in West Berlin last May 29–Jun. 3, attracted the attendance of more than 1,200 participants, representing 36 nations. They heard reports on seven topics of international interest: disinfection, pollution, metering, statistical analysis, automation, local and regional supply systems, and large-diameter pipe. Two plenary sessions were devoted to the subjects of desalinization and radioactivity, and there was a special report on the new activities of the World Health Organization. Manufacturers were present also, and the exhibits that seemed to attract the most attention were those featuring large-diameter pipe, of concrete, steel, or asbestos-cement.

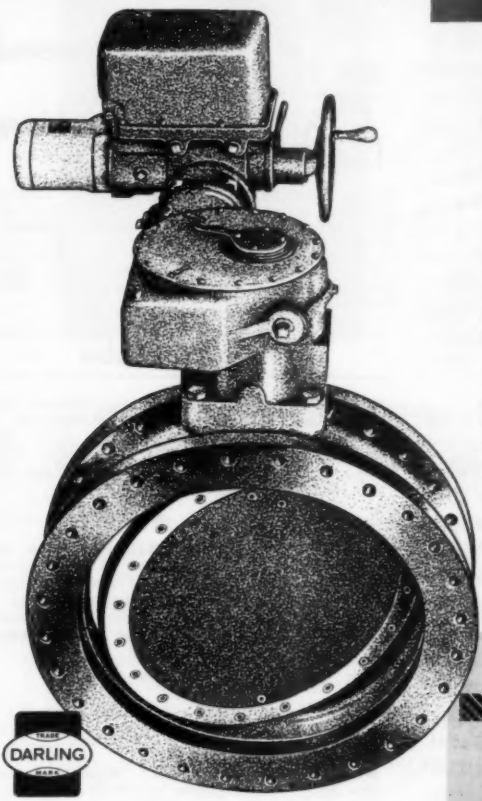
Richard V. Ford, Fred Merryfield, and Remig Papp represented AWWA as observers, with Richard Hazen and Alex Van Praag Jr. as alternates. The next congress will be in 1964, at Stockholm, Sweden.

Willard T. Chevalier, former director of McGraw Publishing Co., died Jun. 29 in New York City, at the age of 75. Born in Brooklyn, he was

(Continued on page 52 P&R)

YOU CAN DEPEND ON

Darling



GATE • BUTTERFLY • CHECK • SPECIAL VALVES • FIRE HYDRANTS

drop-tight shutoff
maximum seat life
easy operation

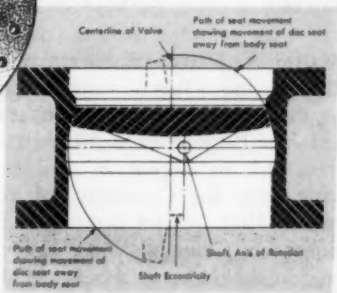
Darling-Pelton Butterfly Valves have a continuous 360° rubber sealing ring, uninterrupted by the valve shaft, that makes complete drop-tight shutoff possible. The valve disc, swinging on an axis eccentric to the valve centerline, is exceptionally easy to operate. It lifts away from the body seat on opening to eliminate any abrasion or distortion. The rubber seat can be replaced quickly without dismantling the valve. *You can depend on* Darling-Pelton Butterfly Valves for safety, long service, superior performance.

Write, wire or phone for your copy of Bulletin 5904 giving complete information on these butterfly valves made to A.W.W.A. Specifications.

DARLING VALVE & MANUFACTURING CO.
Williamsport 23, Pa.

The Canada Valve & Hydrant Co., Ltd., Brantford 7, Ontario, Canada
Vannes Darling-France, 23 rue du Commandant Mouchotte, St Mandé, France

Exaggerated diagram showing eccentricity of valve shaft in relation to valve axis and how the disc, upon opening, is lifted from the body seat throughout its entire circumference.


Centerline of Valve
Path of seat movement showing movement of disc seat away from body seat
Path of seat movement showing movement of disc seat away from body seat
Shaft, Axis of Rotation
Shaft Eccentricity

(Continued from page 50 P&R)

graduated from the Brooklyn Polytechnic Institute with a degree in civil engineering. Prior to World War I, he worked as an engineer on various industrial and utility projects, including the New York City subways and the state's barge canal. He served in the Army Engineer Corps in World War I, afterwards becoming a colonel in the reserves. After serving as general manager of Wailes-Dove Hermiston Corp. (now a division of the Koppers Co.), he joined McGraw-Hill in 1922 as associate editor of *Engineering News-Record*. He became business manager and then publisher of that magazine, as well as publisher of four other McGraw-Hill publications. In 1934 he was selected as a director of the company, a position from which he retired earlier this year. He was named executive assistant to the president in 1945, executive vice-president in 1950, and vice-president and executive assistant to the president in 1955, which position he held until 1957.

A member of AWWA since 1934, he was awarded the Goodell Prize that year and served as a director in 1937–40. He was selected as an Honorary Member in 1945. Other professional affiliations included ASCE, the Society of American Military Engineers, and WSWMA (past-president).

Walter A. Peirce, former manager of the Racine (Wis.) water department, died on Jun. 27 in Racine, at the age of 73. Born in Polo, Ill., in 1888, he received his degree in civil engineering from the University of Wisconsin in 1912. After serving as city engineer in Columbus and Delavan, Wis., and engineer at a sewage plant in Madison, Wis., he went to Racine in 1928 to become manager of the water department, the post he held until his retirement in 1957.

A member of AWWA since 1922, he served as chairman of the Wisconsin Section and twice as director (1932–34 and 1939–40). He received the Fuller Award in 1942 and was a Life Member of AWWA, as well as of ASCE.

Employment Information

Classified ads will be accepted only for "Positions Available" or "Position Wanted." Rate: $1.50 per line (minimum $5.00), payable before publication. Deadline for ad copy: first of month prior to month of publication desired. To place ad, obtain "Classified Ad Authorization Form" from: Classified Ad Dept., Journal American Water Works Assn., 2 Park Ave. New York 16, N.Y.

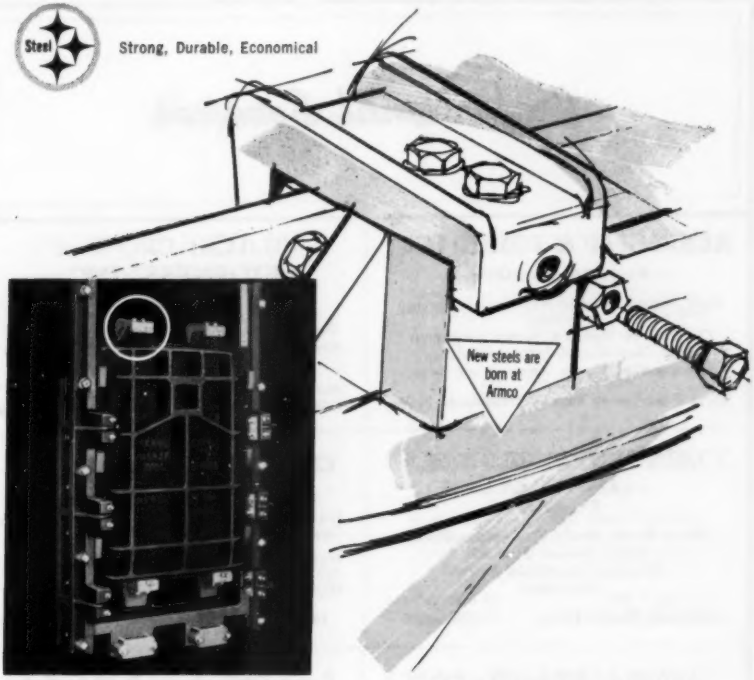
Positions Available

ENGINEER, Design and Research. Graduate Sanitary or Civil Engineer, with 5 years' experience in water works or allied field. Permanent position. Give experience, training, and salary requirements. Write Personnel Office, City Water Board, Post Office Box 2449, San Antonio, Texas.

Water Utility District Manager. Man with experience in the administration of a commercial office as well as field and plant operations. Salary range $7,500–9,000. An excellent opportunity to become associated with one of the fastest growing counties in the country. Interview will be arranged on the basis of resume submitted. Write:

Suffolk County Water Authority
P. O. Box 37
Oakdale, Long Island, New York

MUNICIPAL ENGINEER. Town of Boonton, New Jersey. Requirements: (a) Public water supply system license, Group 3; (b) Degree in civil or professional engineering; (c) New Jersey professional engineering license; (d) Minimum of four years of professional engineering, including at least two years of comprehensive municipal experience. Duties: Supervisor of public works, including water department, streets and roads, and general municipal engineering. Salary: Open, commensurate with experience and ability. Submit resume to Mayor and Board of Aldermen, 210 Main Street, Boonton, New Jersey.



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Detail on Armco's top wedging device for water control gates shows how Armco Gates are built for lasting strength. Note that the reinforced top wedge block is held in position by *two* cap screws for extra rigidity. Further, the adjusting screw, tapped through the back leg of the wedge block, bears against the solid cast iron boss on the casting rather than against another fastener. These three screws act independently to provide a *durable, watertight* installation.

Find out more about Armco Water Control Gates. Write to Armco Drainage & Metal Products, Inc., 4981 Curtis Street, Middletown, Ohio.

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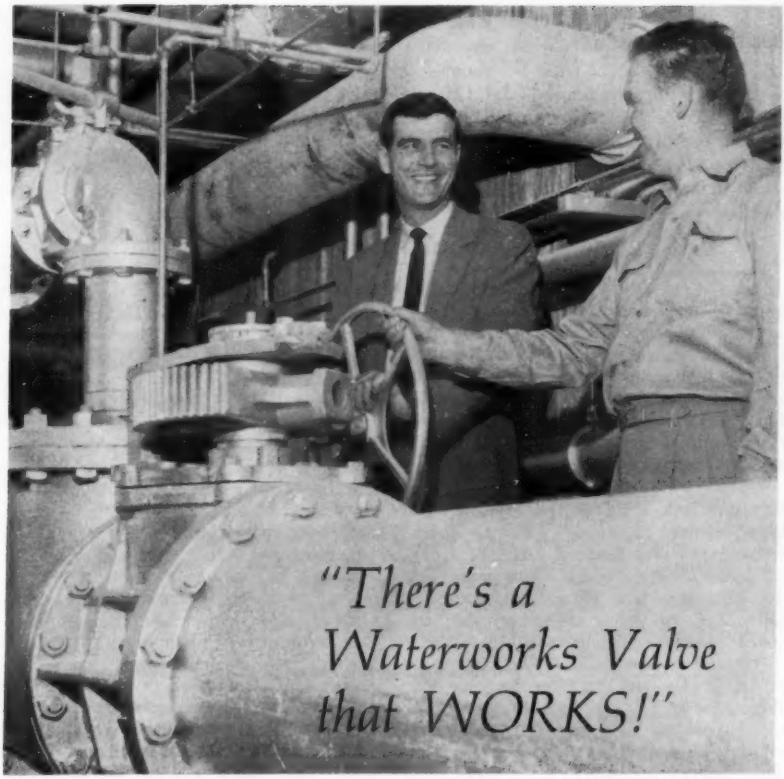

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Key: In the reference to the publication in which the abstracted article appears, 39:473 (May '47) indicates volume 39, page 473, issue dated May 1947. If the publication is paged by the issue, 39:5:1 (May '47) indicates volume 39, number 5, page 1, issue dated May 1947. Abbreviations following an abstract indicate that it was taken, by permission, from one of the following periodicals: *BH—Bulletin of Hygiene (Great Britain); CA—Chemical Abstracts; Corr.—Corrosion; IM—Institute of Metals (Great Britain); NSA—Nuclear Science Abstracts; PHEA—Public Health Engineering Abstracts; SIW—Sewage and Industrial Wastes; WPA—Water Pollution Abstracts (Great Britain).*

FILTRATION

Filters of Deposited Materials. F. A. Beutler. *Gas- u. Wasserfach.* (Ger.), 100: 199 ('59). The author describes the construction of filters by deposition of the filter layer from water on a carrier. Suitable materials for the carriers and methods of insuring satisfactory formation of the deposited layer are described.—*WPA*

Effect of Tank Effluent Quality on Percolating Filter Loading. E. J. Hall. *Inst. Sewage Purif. J. Proc.*, Pt. 1, 85 ('60). Two identical percolating filters were fed, resp., with raw tank effluent and resettled effluent, and composite 24-hr samples analyzed for O absorption from 0.013*N* $KMnO_4$ in 4-hr and 5-day BOD. The degree of purification in a trickling filter does not bear a direct linear relation to the strength of the tank effluent applied. Improved sedimentation is shown to be capable of giving a filter loading increased by 15–20%.—*CA*

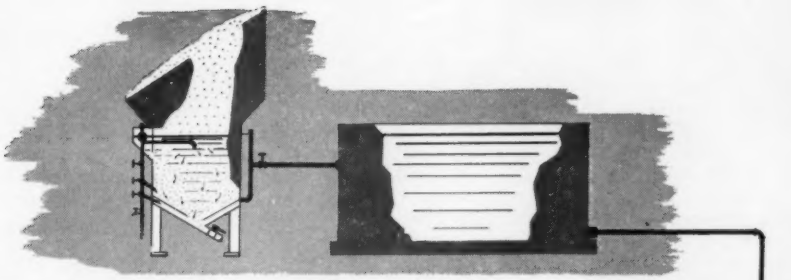
Treatment of Surface and Well Waters and Use of Dolomite Materials for Filtering. H. Borner. *Termotecnica* (Milan), 14:439 ('60). Dolomite materials, calcined below 800°C, can be used with success for filtration and for acidity, Fe, and Mn removal from surface and well waters. Humic acids and humates are also removed with the aid of coagulants. Unlike natural untreated materials, such as gravel and marble, semicalcined dolomite shows an extraordinarily high microporosity, which dets. its very fine filtering power. Besides, it maintains in water an ideal alkalinity which favors the pptn. of Fe and Mn, without clogging the filters. This is particularly valuable because it reduces the need for control at the water-conditioning plant.—*CA*

Some Aspects of Upflow Filtration. F. G. Hill. *J. Inst. Water Engrs.*, 14:505 ('60). The upward-flow filter may be more suited to present-day needs than the conventional downflow filter. The advantages of the upflow filter include: (1) There is no need for controls. (2) The filter cannot be run dry while in use. (3) The filter cannot become air bound. (4) The water passes first through the coarser elements, then through the finer medium. This provides better filtration. (5) Chlorine can be easily and effectively added to the incoming water to maintain a disinfected medium. (6) Some suspended matter settles in the underside of the filter and thus prevents clogging. (7) Changes in color and appearance can be noted, since the filtrate can be observed readily. (8) The various layers of different-size media can be maintained in their proper position, as the finer medium tends to move to the upper layers, where it is more effective in filtration. (9) A more even distribution throughout the medium can be maintained. (10) Breakthroughs are detectable visually. (11) The upflow filter is very flexible; this permits alterations in filtration with little effect on the filtrate. (12) There is a very low loss in head, generally not in excess of 15 in.—*CA*

Filtration After Mutual Coagulation of Sols. S. A. Kretinin. *Trudy Voronezh. Gosudarst. Univ.* (USSR), 57:160 ('59). $Fe(OH)_3^+$ and humus sols, which carry opposite elec. charges, intercoagulate through the whole filtering layer of sand, lowering its filtering property by 99.5%. $(FeOH)_3$ has to be added first, and then the humus. The same colloids, applied in the same order on an artificial soil, stopped the filtration of water through the soil almost completely (99.9%).—*CA*

Mean Residence Time of a Liquid in a Trickling Filter. M. D. Sinkoff; R. Porges; & J. H. McDermott. *Proc. ASE, J. San. Eng. Div.*, 85:51 ('59). Pilot plant

(Continued on page 64 P&R)



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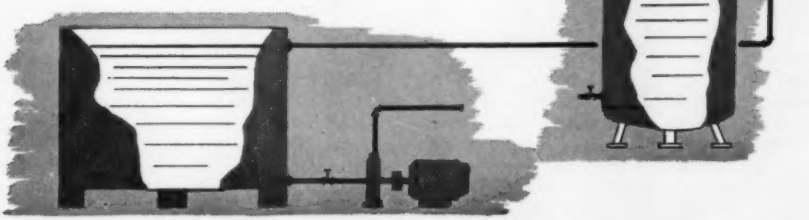

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(*Continued from page 62 P&R*)

research studies aimed at broadening the basic knowledge relative to the performance of trickling filters were initiated. This paper presents the results of initial hydraulic studies in which the mean residence time was detd. for tap water applied to columns packed with spherical media under conditions of varying hydraulic loading rates, media size, and depth.—*PHEA*

Some Experiences With Filters, Old and New. L. DAWSON. *Inst. Sewage Purif. J. Proc.*, Pt. 1, 23 ('60). Observations of the performances of coarse-graded filters in small sewage plants are reported in detail and show that most of the filtration occurs during passage through the first foot of this type of filter and that a depth of 3 ft, with 2 ft of 1-in. grade and 1 ft of 0.5-in. grade, is adequate for a high degree of removal of detergent.—*CA*

Mixed Filters for Demineralization [of Boiler Feedwater]. P. PRACHT & H. BUSKENS. *Energie*, 12:518 ('60). Mixed-bed filters are recommended for the last stage of an ion-exchange demineralization process, so as to remove salt and a SiO_2 cont. of 4 mg/l in natural H_2O; after 15 hr of operation, the SiO_2 cont. of the treated water exceeded the normal value (0.05 mg/l) and after 30 hr it was 5 mg/l (higher than the initial value). With a SiO_2 cont. of 40 mg/l in the H_2O, after approx. 47 hr of operation, the SiO_2 cont. of the treated water was 80 mg/l. Elec. cond. is not a reliable indicator of the point at which the ion exchange should be regenerated; this should be detd. by the SiO_2 cont. of the treated water.—*CA*

Deaeration of Pores in Gas-Saturated Sands, and Study of Their Filtration Properties. Y. L. AVETIKYAN. *Izvest. Akad. Nauk Armyan. S.S.R., Ser. Tekh. Nauk*, 11:61 ('58). A description of the filtration app. and a detailed cross-sectional diagram are given. The potable H_2O satd. with gas is preliminarily heated to 40°C and allowed to flow through the condenser to cool to 13–16°C. Change in temp. provokes some deaeration of H_2O; subsequent passing through the filtration app. in which there exists a pressure of 0.2 atm. allows the H_2O to dissolve the gases contained in the sand pores. After hours of continuous filtration, the filtration coeff. becomes const.—*CA*

Controlled Filter Breakthrough by the Addition of Activated Silica to Settled Water. A. T. PALIN. *Wtr. & Wtr. Eng.*, 63:258 ('59). During water filtration, alum breakthrough occurred long before there was any appreciable headloss across the filter. The addn. of small amts. (0.25 ppm) of activated silica after sedimentation but prior to filtration produced water of low color, turbidity, and alum cont., and greatly increased the filter run time.—*CA*

HYDROLOGY, CONSERVATION, AND IRRIGATION

Marine Studies in the Inshore Waters of Antarctica Near Mawson, 1956. J. S. BUNT. *Nature* (London), 183: 1541 ('59). The author gives results of hydrologic and biologic investigations on the inland waters of Antarctica near Mawson. Weekly observations were made at various depths from Jun. 1956 to Feb. 1957, the surface being frozen for the major part of this period. The chlorinity values obtained during winter were low, mean figures ranging from 16.00 to 18.70 per thousand. These chlorinities, combined with mean temp. between −1.90°C and −1.60°C, commonly produced freezing conditions extending well down into the water column. Chlorinities were appreciably higher during summer, except for a temporary shallow surface zone of extremely low salt concn. produced by the melting of the sea ice. Evidence was obtained of a process by which the summer waters undergo incomplete mixing and remain close to the coastline until their replacement by relatively homogeneous water of higher chlorinity at the beginning of the following summer. The incidence of submarine solar radiation was found to exhibit marked seasonal differences, being negligible during the long winter period. Consequently, phytoplankton were virtually absent during the winter period, though a few zooplankton were found. Large-scale blooms did not occur until the ice was in an advanced stage of disintegration during the thaw. Based on chlorophyll concns., it was estd. that the carbon cont. in the standing crops of summer phytoplankton was between 0.30 and 14.80 mg/cu m.—*WPA*

Hydrology of Tritium in the Ottawa Valley. R. M. BROWN. *Geochim. et Cosmo-*

(*Continued on page 66 P&R*)

RED WATER?
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(*Continued from page 64 P&R*)

chim. Acta, 21:199 ('61). The deposition of T in rain water at Ottawa is reported for 1953–1959. The anal. method involved enrichment of heavy isotopes of H in water samples by electrolysis, redn. of the concentrate to H gas over hot Zn, and counting T in a low-background gas counter. The previous estimate of the cosmic-ray T level in Ottawa rain water has been revised to 15.3 T units (1 T unit = 1 T atom/10^{18} H atoms). It is estd. that the T cont. of Ottawa River water in the pre-bomb period was about 15 T units. A hydrodynamic model for the Ottawa system is developed mathematically on the basis of correlating the T cont. of rain and river waters. A turnover rate const. of 0.27/yr for a storage reservoir of 2.1 m was calcd. The fraction of fresh pptn. in re-evapd. water was estd. at 0.55 ± 0.1. The deposition rate of cosmic T was detd. to be 1.32 ± 0.10 atoms/sq cm/sec. Bomb T deposited by the end of 1959 was 7.3 times the natural T inventory. Correlations provided by the model are useful in estg. turnover rates in other nearby water systems.—*CA*

Hydrogeology of the Upper Carboniferous and Rotliegend Formations of the Western Erzgebirge Basin. H. J. Bluher. *Geologie* (Berlin), 9:909 ('60). Some anal. of the water from Zwickau-Oelsnitz and their further surroundings are given. The drinking water economy of this area and the water conditions of the Zwickau-Oelsnitz coal district are discussed.—*CA*

Discharge From Heavy Rainfall. E. E. Dawson. *Proc. Inst. Civ. Engrs.* (London), 10:453 ('58). A simplified catchment is used to obtain the hydrograph resulting from rainfall of an intensity varying in time and area. The hydrograph for intensity uniform in time and area is obtained mathematically, and a graphical method is used for other variations of intensity. The actual hydrographs for five storms on the Damodar catchment area, India, are reproduced from the known rainfall by the graphical method, and from the results the maximum discharge likely to occur in this catchment is predicted. —*WPA*

Use of a Radioactive Isotope in Hydrology. H. Moser & F. Neumaier. *Gas- u. Wasserfach.* (Ger.), 100:657 ('59). I^{131} was added to a stagnant pond (oxbow) to det. whether water from this pond contmnd. a well. The amt. of I^{131} added was considerably below the permissible tolerance. It was found that such contmn. could occur under certain conditions, but that the contmn. decreased with continued pumping.—*CA*

Composition of Irrigating Waters From Reservoirs East of the Tisza River (Hungary). M. Dzubay. *Agrokemia es Talajtan* (Hungarian), 8:251 ('59). The compn. of irrigating waters, originating from the Hortobagy and Berettyo Rivers, was studied in relation to the qual. of soil in the river bed, in the irrigation canals, and in the reservoir beds. The original compn. of the river waters has been reported. The sp. cond. of the waters was in relation to the solids (dissolved salts) cont. The compn. of the irrigation waters will not differ significantly from the compn. of the original waters (even if there are alk. soils involved) provided that the canals are at least 2 years old, that there is no ingress of ground water into the canals, and that the waters are in continuous motion (5–50 cm/sec/min) in the canals.—*CA*

Evaporation Control Research, 1955–58. R. R. Cruse & G. E. Harbeck Jr. *USGS Wtr. Supply Papers,* No. 1480 ('60). Homologous straight-chain fatty alkanols, e.g., dodecanol, are the best evapn. retardants. Wick-type drippers are best for applying liquids; and cage rafts are best for applying solids. Biochem. oxidn. can be reduced by Cu compds., but all are hazardous. Wind action on the monofilm presents a serious problem. Present procedure on larger reservoirs reduces evapn. no more than 20%.—*CA*

The Influence of Woods and Treeless Land on Runoff, Water Storage, and Soil Erosion. I. Delfs et al. *Literaturber. Wasser, Abwasser, Luft & Boden* (Stuttgart), 8:6 ('59). Details are given of investigations between 1949 and 1953 into rainfall, runoff, evaporation, retention of water, and soil erosion in 5 areas in the Harz Mountains. The areas examd. lie in the drainage region of the Oker and were mainly wooded, but in part had been completely deforested between 1945 and 1948. Figures are given for retention of rainfall by pine

(*Continued on page 68 P&R*)

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(*Continued from page 66 P&R*)

woods of different ages, for runoff from wooded and bare ground, and for erosion. Though such figures can apply only to the region examined, their value for purposes of comparison is pointed out.—*WPA*

Reducing Evaporation With Cetyl Alcohol Films: A New Method of Treating Large Water Storages. R. G. VINES. *Australian J. Appl. Sci.*, 11:200 ('60). In calm weather, rapid and complete monolayer coverage of a 300-acre Australian reservoir was best achieved by the dusting for 1 hr each day of 10–30 lb of powd., cool, dry, com. hexadecyl alc. (contg. 50% octadecyl alc.) from a "grinder-duster" machine on a moving boat. —*CA*

Determining Runoff From Rainfall. J. E. NASH. *Proc. Inst. Civ. Engrs.*, 11:163 ('58). Various methods of detg. the relation between effective rainfall and storm runoff are discussed and shown to be particular cases of the general unit-hydrograph theory. A systematic approach to the investigation of the relation between the characteristics of a catchment and its response to rainfall is indicated. This method is being applied to a series of natural catchments.—*WPA*

Thermal Formation Waters in the Paleozoic and Mesodoic Sediments of the Western Crimea. M. M. GERMANYUK. *Sovet. Geol.* (Moscow), No. 6, 111 ('60). Drilling in the Novoselov structure and Sakki elevation of the Western Crimea indicated the presence in Paleozoic and Jurassic strata of highly mineralized thermal formation waters contg. I, Br, NH_3, and naphthenic acids. Owing to the great depth and low outflow, the waters of the Tavrichesk and Al'b-Apt formation of the Sakki elevation have no practical value. The output of formation waters from the Neokomsk series in the Noveselov structure is sufficiently large to use this high I and Br thermal water for therapeutic purposes and for heating of nurseries and houses. The explored area of the water-bearing formation is 300 sq km. On the Sakki elevation the same formation waters are weakly mineralized and can be used as mineral waters. The discovery of this water in the Sakki elevation can solve some irrigation problems of the foothills of the Crimea. Numerous tables and a geoisothermal map of the Western Crimea are given.—*CA*

(*Continued on page 70 P&R*)

Roll-call for water

Indians Say: "Ugh! Too Many White Man!"

In 1814, when Jackson defeated the Creek warriors at Horseshoe Bend, Ala., it probably never had occurred to anyone that someday there would be so many people in the east-south-central States that there could be a shortage of water!

But that Indian defeat was the beginning of a new era. It set the stage for settlement of the Deep South by white people. Immigrants poured into the section, started growing cotton, then corn, then livestock. Later they discovered coal, iron ore and other minerals until sections of this country today support a great industrial economy.

It would have been hard for Andrew Jackson to believe that there ever would be 29 public water systems in this Deep South country, each supplying 25,000 or more population; but that 5 (16%) of the 29 water systems would be of doubtful efficiency and 4 (13.7%) of them would be definitely deficient. Yet, this section of the U.S. is in better shape than the country as a whole where the definite water system deficiency runs 26.6% and the doubtful efficiency totals 23.2%.

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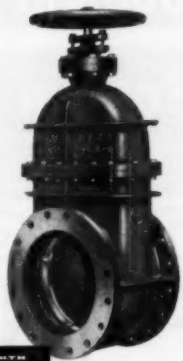

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(Continued from page 68 P&R)

A Method of Evaluating the Effect of a Monomolecular Film in Suppressing Reservoir Evaporation. G. E. HARBECK JR. & G. E. KOBERG. *J. Geophys. Research*, 64:89 ('59). Evapn. from a H_2O surface may be reduced by application of a unimol. film. Because of possible seasonal variation in evapn. pan coeffs., computed savings in evapn. may be unreliable if based on evapn. pan records. A more reliable measure of evapn. redn. is obtained by soln. of equations based on energy budget and heat and mass transfer theory. The rise in H_2O surface temp. caused by application of the film can be computed.—*CA*

Irrigation Waters. II. Some Aspects of Irrigation Waters of Eastern Sicily. E. GIOVANNINI; M. DI GIORGI; & P. FICHERA. *Tecnica agr.*, 10:559 ('58). Data on the chem. compn. of 18 irrigation waters from rivers and 20 from wells (some sampled in different seasons) are reported, with some discussion on treatments to be made for a satisfactory utilization.—*CA*

The Hydrology of the Mount Pleasant-Angaston Area. E. P. O'DRISCOLL & R. I. CHUGG. *Dept. Mines S. Australia, Geol. Survey, Rept. Invest.*, No. 14, 1 ('59). Chem. anals. are given of 168 ground waters, many too saline to be usable for irrigation. —*CA*

Phase Transformation in Commercial Cetyl Alcohols for Water Conservation. R. G. VINES & R. J. MEAKINS. *Australian J. Appl. Sci.*, 10:190 ('59). In the use of monomolecular films for retarding evapn. from water surfaces, some difficulties have been caused by the variability of commercial cetyl alcohols, as many of these differ widely in spreading properties. The authors have therefore studied phase transformation in 2 different types of commercial cetyl alcohol pellets, using dilatometric measurements, heating and cooling curves, and x-ray and dielectric studies. The results indicate that deterioration of the pellets with use may be due largely to phase changes within the pellets, these changes being greatly accelerated

(Continued on page 72 P&R)

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(Continued from page 70 P&R)

by the presence of water. If the pellets become overheated during the day, they can transform to a poorly spreading modification from which the better spreading form will not be regenerated quickly at night unless the temp. falls to improbable levels. Pellets of soft opaque material are inherently superior to hard translucent pellets, as their transition temp. are considerably higher. However, they have the disadvantage that they are too soft and become mushy in water, and in this sense the harder sample is preferable. The behavior of both kinds of pellet would be improved if it were possible to suppress the phase change or raise the transformation temp. considerably.—*WPA*

The Mark VI Ground Water Standpipe for Measuring Seepage Through Salmon Spawning Gravel. L. D. B. TERHUNE. *J. Fisheries Research Board Can.*, 15:1027 ('58). The ground water standpipe is a length of pipe, pointed at one end and with a number of perforations near the point. This is driven into the gravel, where the ground water flows through it. The permeability of the gravel is measured by lowering the level of water inside the standpipe 1 in. below the level outside and measuring the rate of inflow. The apparent velocity through the gravel is measured by introducing a color into the water in the standpipe and detg. its rate of diln. from successive small samples taken at appropriate intervals of time. In the Mark VI model, which is newly designed and recalibrated, the standpipe is open during measurements of permeability. During measurement of ground water velocity, the color diln. chamber is completely sealed, and the color is introduced and samples withdrawn through a sphincter valve with a hypodermic syringe. The soln. is stirred during the period of diln. Details of design and construction of the standpipe and procedure for its use in salmon spawning gravels are appended.—*WPA*

(Continued on page 76 P&R)

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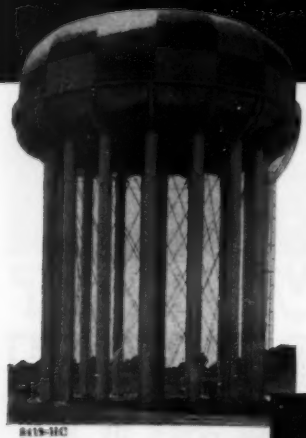

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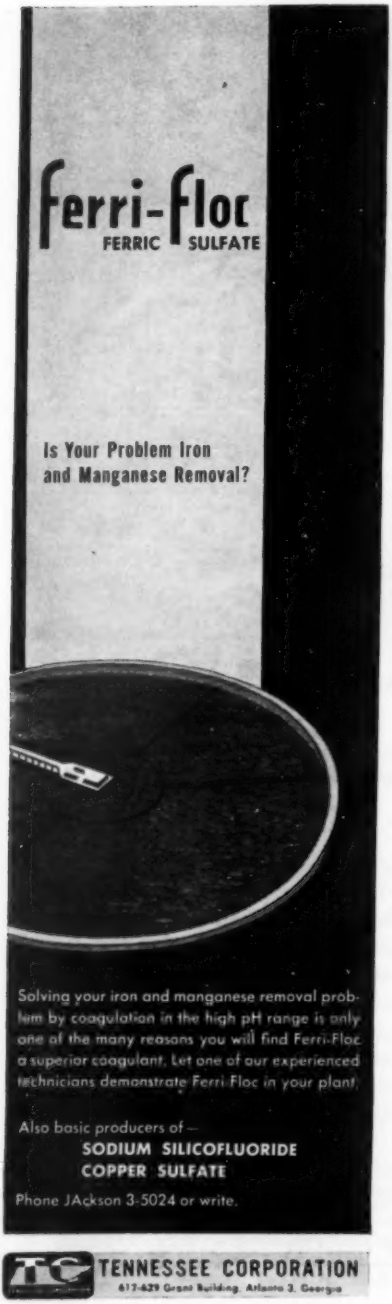
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(*Continued from page 72 P&R*)

FLUORIDATION

Identification of Small Quantities of Fluoride in Waters Rich in Chloride. F. Moreno & M. Vila. *Anales bromatol. Madrid*, 11:185 ('59). A method is described for the detection of fluoride ions as silicon tetrafluoride after preliminary sepn. from chloride ions by treatment with sulfuric acid, followed by distillation and removal of the remaining hydrochloric acid with silver sulfate. The method can detect 0.01–0.1 mg fluoride in the presence of 1 g sodium chloride.—*WPA*

Fluorine Damage to Cattle. H. Reinhard. *Schweizer Arch. Tierheilk.* (Swiss), 101:1 ('59). Atm. F from a factory producing Al is in the form of HF and SiF_4. About 500 kg F is emitted per day. The amt. of F sedimented was 7 kg/hr 300 m from the factory, and decreased to 0.9 kg/hr at 700 m. Plants with large leaves which last a long time, e.g., corn and sugar cane, contain the greatest amts. Grass and hay contained normally 0.1–0.5 mg% F; in some areas it contained >50 mg%. Fodder contg. 5 mg% F caused alterations of the teeth and bones, lameness, emaciation, decrease of milk production, and apathy in cattle. The amt. of F in bone ash and urine of cattle increased from 40 mg% and 0.4 mg/l, to 200–300 mg% and 2–10 mg/l, resp. Videhydrosol, Tonophosphan, and high Ca doses were used for treatment. Therapy with oxytocin was successful to increase milk production.—*CA*

Fluorine Content of the Water Supplies of Municipalities Situated in the Coastal Wooded and Uncultivated Zones of Pernambuco State. M. R. da Costa Carvalho & D. de Melo Fonte. *Anais fac. farm., Univ. Recife* (Brazil), 2:91 ('59). Data on F cont. are given for 40 samples from 31 municipalities; 96% of the municipalities in the coastal wooded area showed 0.4 mg/l (0.4 ppm) or less, with only 1, (4%) at 0.9 mg/l. In the uncultivated area, only 1 place showed as high as 0.5 mg/l, all the others were from 0.0–0.4 mg/l.—*CA*

The Fluoride Content of Dalmatian Drinking Water. D. Tomic. *Gesundheits-Ing.* (Munich), 81:24 ('60). The F cont. of drinking water of the Dalmatian Islands and

(*Continued on page 78 P&R*)

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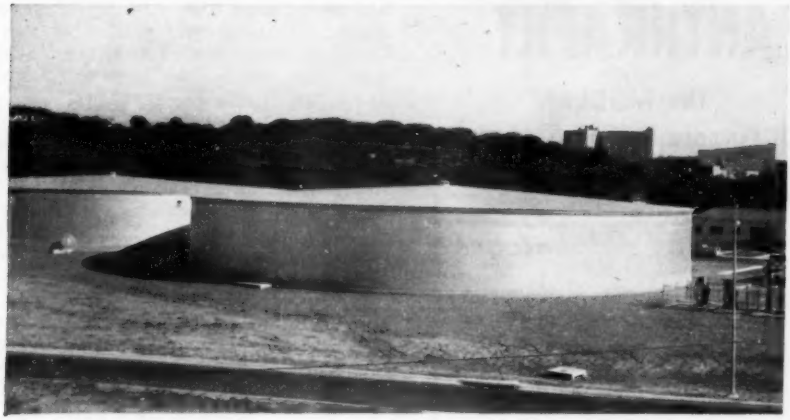

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(*Continued from page 76 P&R*)

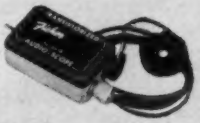
mainland was detd. F concn. on the mainland varied from 0.040–0.245 mg/l; on the islands, the range was 0.015–0.240 mg/l. Cl concn. varied from 4.2–530 and 7.5–360 mg/l, resp., and was not apparently related to F concn.—*CA*

Preliminary Data Concerning the Relation Between the Caries Indexes and the Fluorine Concentration of Drinking Water in the Region of Hunedoara. V. POPESCU ET AL. *Stomatologia*, 6:31 ('59). Of 19 sources of water investigated, 8 contained less than 0.350 mg/l; 7, 0.350–0.650 mg/l; and 4, above 0.650 mg/l. In general, a parallelism was found between the reduced concn. of F and the high indexes of caries of the pop. There is, however, also contradictory evidence, where the F concn. is high and the caries index is high also.—*CA*

Monitoring of Fluoride Content of Air, Water, and Vegetation. C. R. MCHENRY & H. CHARLES. *J. Air Pollution Control Assoc.*, 11:66 ('61). The phosphate fertilizer industry is one of several that emits gaseous fluorides. There are 9 major companies in the Polk Co., Fla., area engaged in the manuf. of various phosphate fertilizers including a triple superphosphate (monocalcium phosphate) manufg. plant which emit gaseous fluorides in the form of SiF_4 and HF. Control measures were incorporated in the plant design to reduce such emissions to a min. A 5-yr poln. study was started 1 yr before the plant was built and continued after the plant was in operation to det. the effectiveness of the controls. The study includes continuous monitoring of air and water samples, vegetation surveys, and meteorologic studies. Results were used to det. a F emission rate consistent with the air poln. laws of Florida and to maintain desirable conditions around the plant site. Addnl. control equip. is being installed to accomplish this purpose. Methods of sampling and anal. of various materials for F are given.—*CA*

Fluorine in Drinking Water of Kazakhstan and Endemic Fluorosis. L. N. KREPNOGORSKII. *Zdravookhranenie Kazakhstana* (USSR), No. 1, 28 ('58). The F cont. of 527 sources of drinking water supply in Kazakhstan, which vary from 0.6 to 10 mg/l, are reported. An increased F cont. in water

(*Continued on page 80 P&R*)



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(*Continued from page 78 P&R*)

is principally observed in two zones of the republic: in Central Kazakhstan, and on the foothills of the Transilian Ala Tau.—*CA*

Fluorine-Containing Drinking Water and the Incidence of Fluorosis in the Town of Isparta (Asia Minor). S. I. GURKAN & S. MELEK. *Bul. fac. med. Istanbul.*, 23:363 ('60). The incidence of fluorosis varied widely in different quarters of the town. The amount of F in the drinking water varied accordingly. The water used in fluorosis-affected quarters contained 0.2% F. No fluorosis was found in the parts of the town where water with less than 0.1% F was used.—*CA*

Fluorine in Rivers. G. S. KONOVALOV & O. S. OGURTSOVA. *Gidrokhim. Materialy* (Moscow), *29*:68 ('59). The F cont. of various rivers was investigated as to their suitability as water supply sources, the max. cont. of F for drinking water being 1.5 mg/l. Samples were taken (usually from the lower reaches) at the periods of max. and min. flow. The thorium-alizarin colorimetric method was used after distg. the F as fluosilicic acid. This gave an error of 0.01 mg/sample (consisting usually of 100–400 ml of water). Concns. from <0.1 mg/l up to a max. (in the southern Bug) of 0.68 mg/l were recorded, the preliminary estimates of total F carried by USSR rivers being 334,188 tons/yr. The northern rivers, fed principally from the frozen north, have lower F content than those where climatic conditions are favorable to higher flows, esp. as the heavy northern rainfall has dissolved out the more sol. F-contg. minerals. The avg. F cont. of the river Angara is attributed to a corresponding cont. in Lake Baikal. The rivers Ponoi and Zeya with 0.17 and 0.13 mg/l, resp., were notable for their relatively high F cont. compared with overall low mineral cont. Generally, there is little variation in F cont. each year or from year to year. Tables are given showing F cont.

(*Continued on page 84 P&R*)

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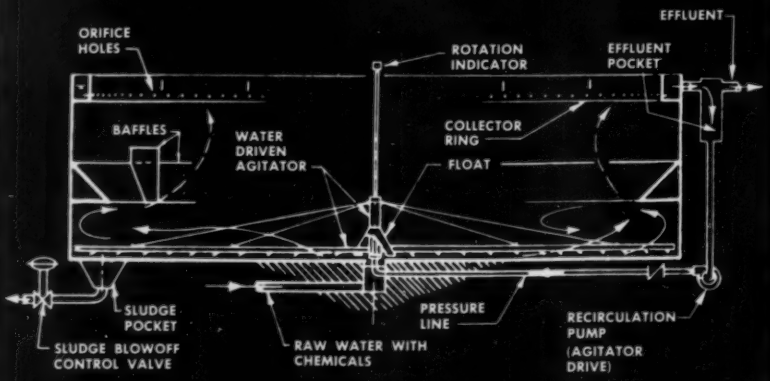

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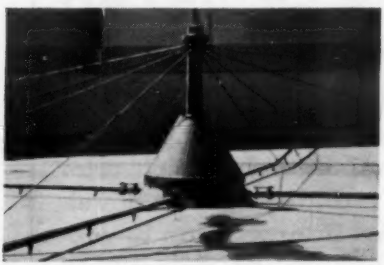

Agitator float and hub assembly.

Briefly, this is how the M unit works: raw water and chemicals are mixed in line and enter at the center of the tank floor beneath the conical rotor float which deflects the fluid mixture radially outward to the walls. A horizontal baffle on the tank wall then reverses the flow back toward the center thus imparting a rolling action to the sludge blanket. This current flow constantly places previously formed sludge in intimate mixture with the raw water immediately as it enters the tank. Clarified, effluent is collected by a flume at the top.

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(*Continued from page 80 P&R*)

of the principal rivers flowing into the Baltic Sea, the Black Sea and Sea of Azov, the Caspian Sea and Sea of Aral, the Barents and White Seas, the Kara Sea, the Laptev, East Siberian, and Chukotsk Seas, and the Pacific Ocean (Bering, Okhotsk, and Japanese Seas).—*CA*

Genesis of Fluorine Water. J. VRBA. *Vestnik ustredniho ustavu geol.* (Prague), 34:365 ('59). The waters can be classified according to their analyses and the geol. formations in which they occur. The underground waters of the Massif of Central Bohemia show a F analysis of 0.2–0.5 mg/l; not over 10% of the surface waters shows more than 2, and only 1 river shows 6.6, mg/l F. These waters are found in Cretaceous formations such as marls and marlstones. In the Bohemian Carpathian system, the underground waters contain up to 0.3 mg/l F; no more than 3% of the surface waters contains over 2 mg/l F, and the max. value found at 1 brook is 3.5 mg/l F. They occur in Triassic-Quaternary limestones and travertines.—*CA*

The Fluorine in the Drinking Water of the Uzbekistan. I. A. Z. ZAKHIDOV & V. E. NAZYROVA. *Med. Zhur. Uzbekistana* (Moscow), No. 10, 65 ('58). The avg. amt. of the F (in mg/l) was 0.6 in the surface water (rivers, channels for irrigation, springs), 0.3–0.5 in most wells, and 0.1–0.3 in the sources (reservoirs) of the drinking water of the city of Tashkent. The amt. of the F reached 2 only in some of the wells of the Samarkand region (Narpaisk district).—*CA*

FOREIGN WATER SUPPLIES—GENERAL

The River Tavy Water Scheme of the City of Plymouth. *Wtr. & Wtr. Eng.*, 63: 501 ('59). The Plymouth Water Undertaking is one of the oldest municipal enterprises in England. Corporation records show

(*Continued on page 86 P&R*)

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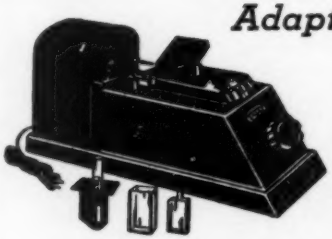

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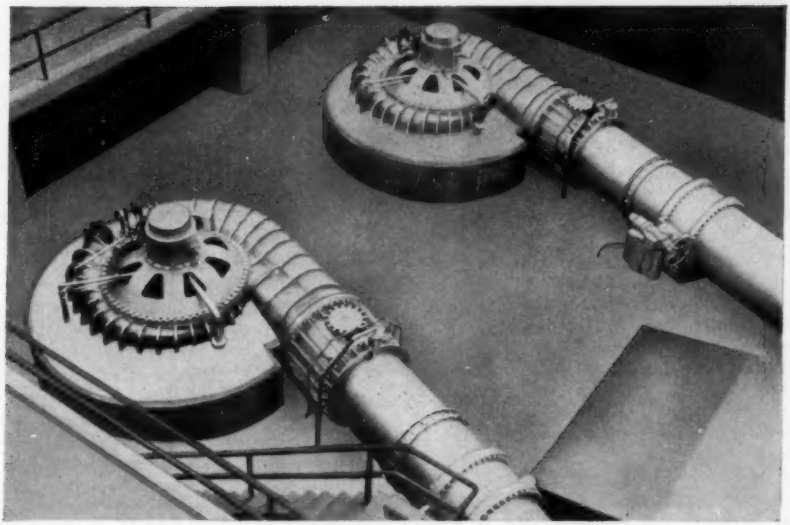

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(Continued from page 84 P&R)

the expenditure of public funds in 1495 for maintenance and repair of wells, pumps, and conduits. The original surface supply which was used to augment the well supply came from diversions on the River Meavy. Increased demands resulted in the construction of the Burrator Dam and Reservoir and its enlargement in 1928. Present consumption is 15 mgd, which exceeds reliable yield by 3 mgd. Although the reservoir is small for the size of the watershed, use of water from the adjoining River Tavy proved more economical than additional storage on the Meavy. A 300-ft long, 10-ft high rubble rock dam was constructed in the tidal part of the River Tavy at Lopwell. Water cut off is provided by sheet steel piling which keeps tidal water out of the fresh water above the dam. A fish ladder permits passage of game fish. Winter excess water from Burrator is piped to Lopwell to generate electricity for which a credit is given. During the summer, this "credit" power is used to pump water from Lopwell to the treatment works above Plymouth. Tavy water is coagulated, settled for 3.25 hr, and pumped to rapid gravity filters. The better quality Burrator water is merely coagulated in a reaction tank which provides 30-min detention. Filters are operated at 1.6 gpm/sq ft. Filters are cleaned by backwashing plus air scour. Lime is added after filtration. Finished water is stored in two covered reservoirs having a combined capacity of 6 mil gal. A small portion of the area is still served raw water from an open reservoir. This condition will be corrected within 2 years.—*PHEA*

Conurbation Water Supplies in Great Britain. S. Grerogy. *J. Brit. Waterwks. Assn.*, 41:72 ('59). The aggregation of urban districts into conurbations in several areas in Great Britain (namely, Greater London, southeast Lancashire, west Midlands, west Yorkshire, Merseyside, Tyneside, and central Clydeside), and the water supplies of these areas, their water demand assessed against the pop., the collection and distrib. of water, and general admin. are discussed. The possible centralization by further amalgamation of water authorities within these areas, the administration this would involve, its associated problems, and future water supplies and demands are outlined.—*WPA*

Examples of Water Use in Closed Circuits. I. At the Sidelor Factory at Micheville. E. Cornuat. *Eau* (Paris), 45:279 ('58). A complete description, with flow charts and tabular data, is given of the water system at the Micheville Works where 94% of the water is recycled. Water resources are described, as are the recycling systems and uses for which the flow systems have been developed.—*PHEA*

Quality of River Water and Its Pollution. VI. The Rivers Maruyama, Yada, and Kishida. S. Okada & K. Uesugi. *Himeji Kogyo Daigaku Kenkyu Hokoku* (Japan), No. 10, 64 ('58). The amt. of total solids, especially that of inorg. salts, was large in the water of the River Maruyama because of the waste water from a metallurgical refinery near the source of the river and a mine on the upper stream. But the amt. decreased gradually in the water of the lower stream. The water was harder, and its amt. of MgO was larger than that of other rivers in the Hyogo Prefecture. The value of MgO:CaO was 2 or 3 times that of other rivers. The water of the Yada and Kishida was comparatively less hard, and total solids were less than those of the Maruyama. The value of MgO:CaO was higher than that of the rivers flowing into the Inland Sea.—*CA*

Distribution of Microelements in the Red-Tinted Waters of Cheleken. Z. Y. Kravchinskii. *Izvest. Akad. Nauk Turkmen. S.S.R.*, No. 1, 115 ('59). The Cheleken stratum (in western Turkemenia) contains reddish waters which were analyzed for alky., borate, I, and Br. I in all samples varied between 23 and 34 mg/l. Br varied between 25 and 572 mg/l, being more concd. in the upper 500 m of the 2500-m thickness studied, and more concd. in the western than in the central Cheleken. Alky. varied in the range of 0.3–43.5 meq/l, while boric acid ranged from 6.1–73.0 mg/l, both tending to increase with depth.—*CA*

Chemical Studies on the Water Accompanying Natural Gas in Niigata City. V. Relation Between the Dissolved Substances and Depth. J. Itaya. *Nippon Kagaku Zasshi*, 80:145 ('59). Chem. anals. are given of 34 samples of water present in

(Continued on page 88 P&R)

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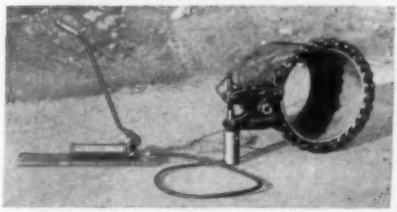

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(*Continued from page 86 P&R*)

natural-gas layers of different depths (151–1,133 m). These waters show a higher ratio of K/Na and lower ratio of Mg/Ca than does sea water. Ratio of $(Na + K + Ca + Mg)/Cl$ is greater than unity in upper layers, whereas ratio of $(Na + K + Ca + Mg)/(Cl + HCO_3)$ is close to unity in all the layers examd. This suggests that in the upper layers free carbonic acid becomes HCO_3^-, by which cations are dissolved from surrounding strata. There are pos. correlations among org. C, HCO_3^-, and the ratio of the amt. of gas to that of water produced in a day. Correlations are also pos. among Cl^-, Br^-, and I^-, and between the above ratio of gas to water and Mg/Ca (with a few exceptions).—*CA*

Physicochemical-Bioclimatological Studies in Wells of Bad Munster am Stein. H. Cauer. *Arch. physik. Therapie,* 10:438 ('58). The mineral waters of the wells of this spa were analyzed for their anion and cation cont. and radioactivity, as well as for temp., pH, odor, and color. The gases used in the emanatorium were analyzed for ammoniacal N and methane. The gas is slightly radioactive. Comprehensive tables of analytic results are included.—*CA*

Radon Content of Cold Waters From Certain Regions in France. M. Roubault; R. Coppens; & G. Jurain. *Compt. rend.* (Paris), 248:715 ('59). By using a scintillation detector for α rays in a mobile lab., the Rn cont. of 344 water samples from springs and wells in certain regions of France was measured in the summer of 1958. Except in mountainous districts, Rn was present in quantities greater than the recognized tolerance for continuous intake, 40×10^{-10} c/l, in more than two-thirds of the samples, regardless of the presence or absence of U in the vicinity. The lower Rn cont. of waters from mountainous areas are ascribed to loss of the element from these rapidly flowing streams. The authors question the value of the recognized tolerance figure for a continuous dose of radon, as the waters examd. had been used for humans and animals for very long periods.—*CA*

The Role of Hydraulic Engineers in Water Supply Problems of French Overseas Areas. M. J. Rodier. *Centre belge etude et document. eaux, Bul. mens.,* 3:804 ('58).

(*Continued on page 90 P&R*)

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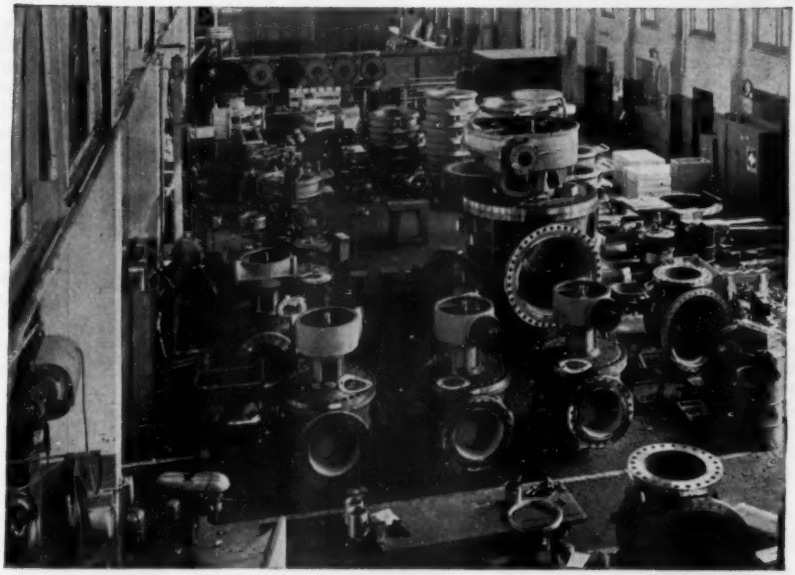

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(*Continued from page 88 P&R*)

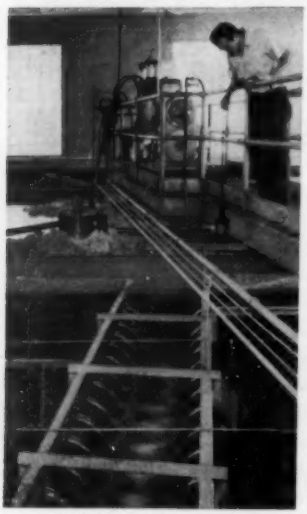
Problems relating to potability, adequate flow, and source are considered in general terms for islands, such as the Antilles, and desert areas, such as the Sahara. A table is presented summarizing some of the problems found in North Africa.—*PHEA*

Technical and Social Aspects of Water Purification in French Black Africa. M. MERLIN. *Centre belge etude et document. eaux, Bul. mens.*, 3:212 ('58). Growth of cities and other concn. of people has increased rapidly in the last 10 years. Studies to facilitate prepn. and distrib. of potable water have been concerned with human geography, cartography, and sources of subterranean and surface waters. Although financing of such operations is difficult, increasing supplies of potable water are being made available to important cities. Secondary centers and rural areas are being considered.—*PHEA*

Delivery and Control of Potable Water in Rural Areas. D. BUTTIAUX. *Tech. sanit. et munic.* (Paris), 53:231 ('58). Various problems relating to water purification, storage, and delivery in rural areas, especially in northern France, are considered. Depth of wells and information concerning bacterial contamn. are tabulated for 7 sources. Reservoirs and storage tanks are described and illustrated together with pumping equip. Control of these waters is maintained by const. examn. for mineral, radioactive, and bacteriologic contamn. Treatment is by means of chlorine and ozone.—*PHEA*

The Water Problem in Africa and the Management of Sahara Zones. M. R. TOURNIER. *Centre belge etude et document. eaux, Bul. mens.*, 3:219 ('58). Problems of water supply in desert areas are considered. These include artificial procedures, tapping subterranean streams, evapn. of sea water, use of mountain water sheds, use of estuaries and rivers, use of liquid and ice storage, and stabilization of shore lines of seas. Methods of application to dune oases are reviewed.

(*Continued on page 92 P&R*)



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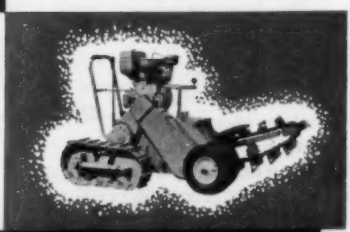

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(Continued from page 90 P&R)

Storage of Niger and Congo River waters is suggested.—*PHEA*

Soluble Inorganic Phosphorus Content in Kieler Fiord, 1952–57. J. KREY. *Kiel. Meeresforsch.* (Ger.), 15:17 ('59). The sol. P cont. of the water of Kieler Fiord was investigated by sampling and analyzing at monthly periods, 1952–57. Samples were taken 1 m below the surface and 1 m from the bottom. Three stations were chosen for sampling. Sol. P is reported in mg/cu m. The min. value obtained 1 m below the surface occurred in 1956 when 8.8 mg was found, with a max. of 21.4 mg found in 1952. The samples 1 m from the bottom showed a very high cont., 50.7 mg in 1955, with a min. of 18.3 mg in 1957. Data are also given for total salt cont. for the same period. The effect of O cont. of the water on sol. P cont. is considered, and results are given corrected for O cont. A detailed review of the literature is included.—*CA*

The Variation of the Salinity of the Water of the Romanian Black Sea and the Shore Lakes of Tekirghiol, Agigea, and the "Lake Without Name." N. I. KALINICHENKO & E. ANTOKHI. *Analele stiint. Univ. "Al. I. Cuza" Iasi, Sect. I* (Bucharest), 3: 287 ('57). The variation of the salinity of the Romanian shore waters is explained by phys. and mech. factors. The presence of H_2S in the Tekirghiol Lake is explained by the decompn. of the org. matter of this lake. —*CA*

New System for the Abstraction of Water for the Towns of Santo Andre, Sao Bernardo do Campo, and Sao Caetano do Sul. W. E. D. OLIVEIRA. *Rev. Dep. Aguas Esgotos Sao Paulo,* 19:41 ('58). A detailed, illustrated description is given of the water supplies and works of Santo Andre, Sao Bernardo do Campo, and Sao Caetano do Sul, in the state of Sao Paulo, Brazil. The total pop. of the 3 towns in 1950 was approx.

(Continued on page 96 P&R)

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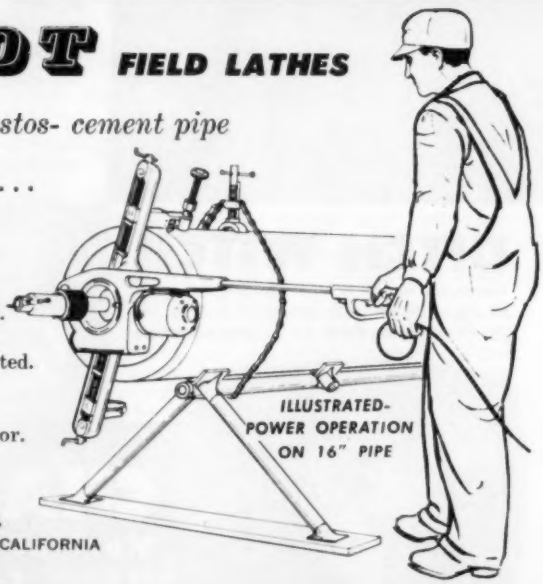

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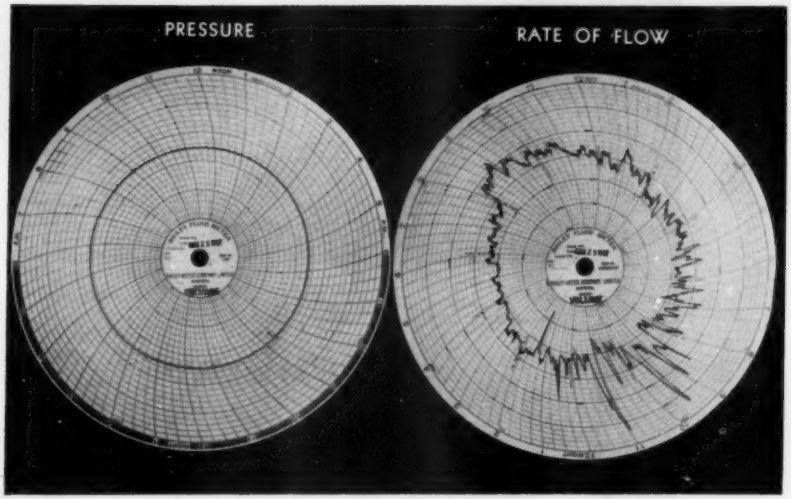

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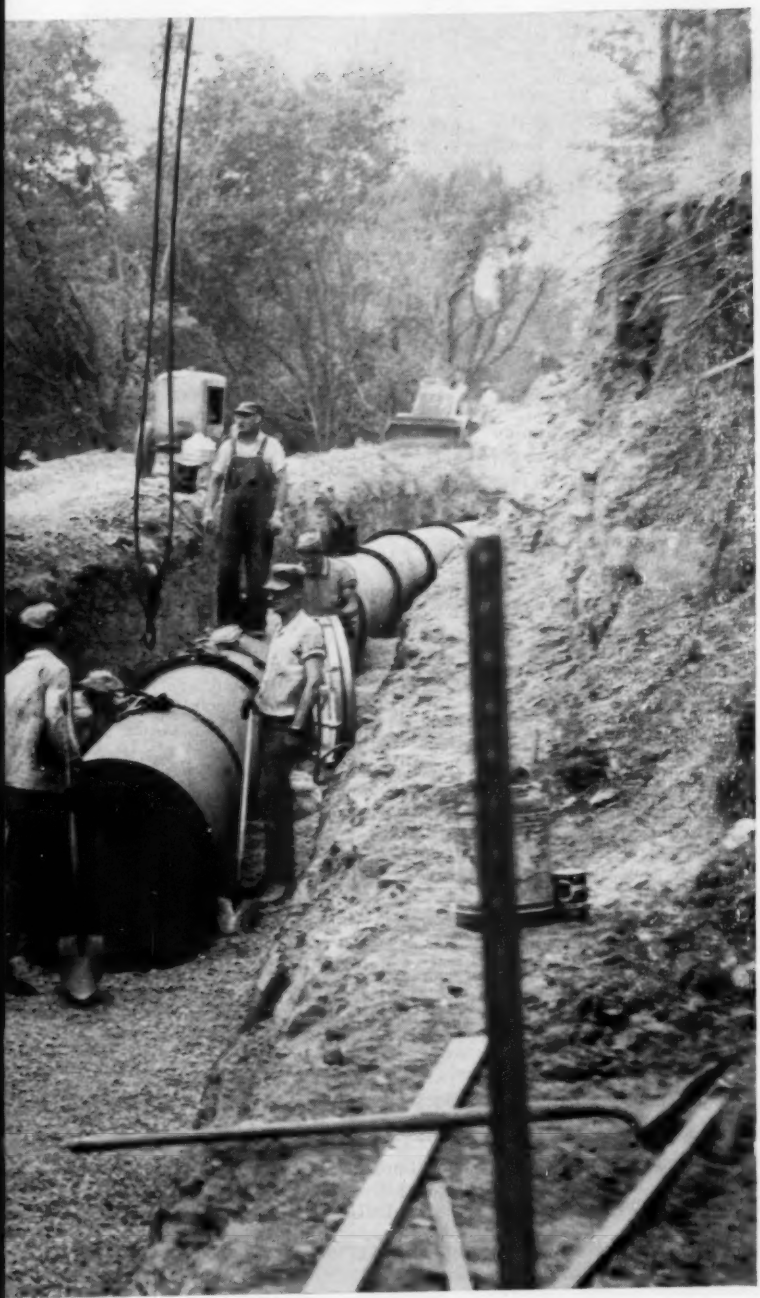


(*Continued from page 92 P&R*)

190,000, and is estimated to be over 500,000 by 1975. The first stage of the new scheme supplies 50,000,000 liters/day, and this will ultimately be increased to almost 173,000,000 liters/day. Water is abstracted from the Rio Grande, a tributary of the Parana River, and is pumped to the treatment plant, where treatment comprises coagulation with lime and aluminium sulfate, sedimentation, rapid filtration, disinfection with chlorine, and adjustment of the pH value. Details of the construction and operation of the plant are given, with the aid of maps and diagrams, and the inauguration of the new scheme, and the terms of the convention under which the scheme was put into effect, are included.—*WPA*

Provision for an Adequate Water Supply in Malta. A. J. SATARIANO. *Sanitarian* (London), 68:163 ('59). In a paper presented at the 1959 annual conference of the Association of Public Health Inspectors, the author discussed the provision of an adequate water supply for the island of Malta. He stressed the importance of such a water supply, gave a brief description of the geology of the island, described the sources of supply, and discussed the demand for water, including industrial and agricultural requirements. The principal source of supply at present is ground water replenished by infiltration of rain water.—*WPA*

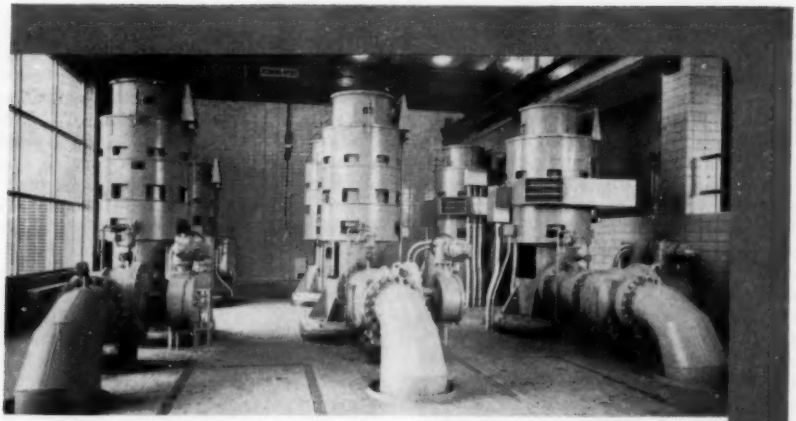
River Waters of Southern Asiatic Countries. I. The Quality of Waters of Thailand. J. KOBAYASHI. *Ber. Ohara Inst. landwirtsch. Biol., Okayama Univ.* (Japan), 11:167 ('59). Complete chem. anals. were made on H_2O samples collected monthly for 1 year from 31 locations in Thailand. Results are given for Ca, Mg, Na, K, HCO_3, SO_4, Cl, SiO_2, Fe, PO_4, nitrate N, ammonium N, albuminoid N, $KMnO_4$ consumed, TDS, pH, hardness, turbidity, and suspended solids. The characteristics of the different waters are related to the rainfall and type of soil over which the H_2O flows. The H_2O of Thailand is compared with that of Japan. It was estd. that in 1 growing season the H_2O would provide the whole rice field area of Thailand with fertilizing substances in the following ton quantities: K_2SO_4 500,000, $(NH_4)_2SO_4$ 100,000, SiO_2 1,300,000, and $CaCO_3$ 5,500,000.—*CA*

Chief Characteristics of Mineral Springs in the Upper Basin of the Agstev River, Armenian S.S.R. R. S. VARTAPETYAN. *Nauch. Trudy Erevan. Gosudarst. Univ., Ser. Geol. Nauk* (USSR), 59:31 ('57). Six mineral springs are located in this area, 5 of which were not previously mentioned in the literature. The geol. structure of the region, the location of the springs, and their specific features are briefly described. Chem. anal. of 5 springs are given. The main components are Ca^{++} (27.2–31.8%) and HCO_3^- (41.9–44.7%); Na^+ and Mg^{++} are present in smaller amts. (4.0–19.2% and 2.6–18.8%); SO_4^{--} and Cl^- are present in minor proportions (1.7–3.7% and 1.8–6.4%). Number 4 spring has 3 mg/l H_2S. The author concludes that the springs have a deep origin and are related to volcanic activity. The appearance of the springs at the surface is related to the Fiolete fracture. The underground location of the waters and the mechanism of their migration are discussed. The waters have a medicinal significance.—*CA*

Management Project for the Dijon Area. *Tech. l'eau* (Brussels), 13:53 ('59). An outline of the development of the water supply for Dijon, France, is given. The town now obtains water from springs in the valleys of the Suzon and Ouche and from 35 wells in the alluvial deposits of the River Saone at Poncey-les-Athee. Water from the Suzon Valley is conveyed by aqueduct to the Darcy storage reservoir and distributed to 148 cisterns; a new storage reservoir, capacity 50,000 cu m, has been constructed at Valmy. The water distribution system has been improved, and Dijon now supplies several neighboring districts with water. Development of the sewerage system is outlined. Sewage was formerly discharged without treatment to the river Suzon, but a sewage treatment plant is now under construction, and an intercommunal sewerage scheme is planned by the Service des Ponts et Chaussees.—*WPA*

Domestic and Industrial Water Supply in the Bonn-Siegburg Area. S. SCHILDER. *Gas- u. Wasserfach.* (Ger.), 99:951 ('58). An illustrated description is given of the construction of an impounding reservoir on the Wahnbach, a tributary which joins the Sieg 4 km above Siegburg. The reservoir

(*Continued on page 98 P&R*)



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(*Continued from page 96 P&R*)

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Ceylon's Water Problems. C. RASIAH & H. C. HUSBAND. *Contract Record & Eng. Rev.*, 71:14 ('60). In a paper presented at a meeting of the Institution of Water Engineers, the difficulties encountered in providing Ceylon with an adequate potable water supply were outlined and discussed. There is no legislation pertaining to water supply or sewage disposal in Ceylon, and, as a result, many wells, from which most of the domestic water supply is still obtained, become pold. Other difficulties are the uneven distrib. of rainfall, which, in some parts of the island, is as much as 200 in. per annum, while in some coastal areas it is only 40 in. per annum. The low pH value of the water and its plumbo-solvency makes lead pipe unsuitable for the conveyance and distribution of water, while the only alternative available is galvanized wrought iron pipe, which is subject to corrosion and incrustation. The few large permanent rivers progress seaward in a series of steps, and are used to provide irrigation for rice fields on these steps. In any case they are generally too turbid for use as a water supply, especially when in flood. In the lower reaches of all the rivers,

(*Continued on page 100 P&R*)

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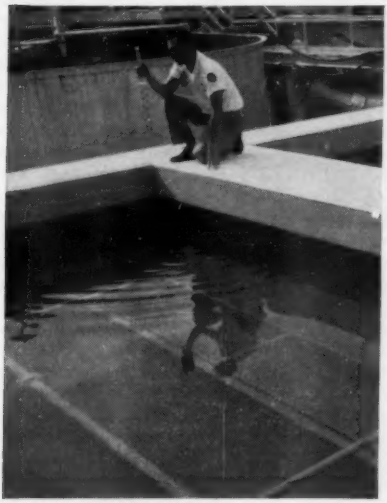

Robert Ankney inspects filter

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The City of Vero Beach, Florida, has a 6 mgd softening plant that draws water from six shallow wells. The plant formerly used activated silica as an aid to coagulation. While results were satisfactory, the plant investigated other coagulant aids in the hope of increasing the efficiency of their operation. After exhaustive laboratory tests and plant trials, Hagan Coagulant Aid #7 was adopted as it is easy to apply, requires less storage space and only a minimum of supervision. A feed rate of only ½ ppm was set up for the Hagan Aid.

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(*Continued from page 98 P&R*)

the salinity of the water is high, which makes it unsuitable for consumption. In spite of these difficulties, a system for supplying treated water to the Colombo municipality is now in operation, and a scheme to supply towns south of Colombo with about 30 mgd of water is envisaged. Some of the major work already carried out on river intakes and on the design of reservoirs and water towers is described in detail, and the new Gal Oya Valley Scheme and industrial water developments for sugar factories is mentioned briefly.—*WPA*

Hydrochemical Conditions of a Few Rivers of White Russia. F. I. SAKOVICH. *Trudy, Belorus. Nauch.-Issledovatel. Inst. Meliorats. i Vodnogo Khoz.*, 8:177 ('58). No corrosive compds. were found in the waters of the rivers investigated. The waters were quite suitable for agricultural as well as tech. purposes. All river waters could be considered soft. The avg hardness was equal to 1–5 German degrees. The waters generally showed an alk. reaction (pH 7.2); in separate cases only, the pH decreased to 6.8. The increased oxidizability of the waters indicated the absence of contamn. by org. compds. HCO_3^- and CA predominated. The cont. of N and K was found to be equal to fractions of g/cu m; the cont. of N was somewhat higher in the contamnd. waters of Neman River. Besides nutritious substances, damaging elements (sulfates, chlorides) for plants were also found; they were brought in by the waste waters from towns and industrial centers.—*CA*

Thermal Spring in Lasko (Slovenia): Geochemical Study. S. MIHOLIC & B. STANCIC. *Croat. Chem. Acta* (Yugoslavia), 31:149 ('59). The present chem. anal. is compared with earlier ones. There are certain changes in the compn., concn., and radioactivity, with a steady lowering of the temp. during the last 100 years. The prevalence of Sn and Cu among the trace metals is indicative of tectonic movements in the Paleozoic. The pH is 8, temp. 35.3°C, radioactivity 1.384 maches/l.—*CA*

Nitrogen Thermal Springs of Northern Transkaikal. V. A. ALBAGACHIEVA. *Izvest. Vysshikh Ucheb. Zavedenii, Geol. i Razvedka* (Moscow), 6:68 ('58). Presented is a study involving, for the first time in this region, the systematic examn. of 126 ground water springs, among which were more than 60 N thermal springs. Also examd. were more than 50 natural discharges of gases. Chem. compns. of the thermal spring waters were detd. in the main by processes of leaching of the granite massifs. The temp. of the N thermal springs fluctuated within the limits of 27–82. The thermal springs had a HCO_3^-–SO_4^{--}–Na^+ compn., slight mineralization, and an alk. reaction. Avg pH ranged from 7.5 to 8.4. It was found that the higher the temp. of the water, the higher the Na^+ cont. in it and the higher the pH. The Na^+ cont. in different springs varied within limits of 46–137 mg/l. Chlorides were present in the N thermal springs in a quantity from 3.4 to 40 mg/l. All the springs contained silicic acid, the amt. of which ranged from 23 to 117 mg/l. F was observed in 32 samples from thermal springs. Its quantity varied from 2.1 to 7.0 mg/l. The thermal springs, as a rule, at their places of discharge yielded deposits of lime. Besides Ca–CO_3, compds. of Si, CaF_2, Al, Sr, Mn, Cu, Ti, Zr, Ga, and Ge also pptd. in the deposit. The N thermal springs evolved gases, and the avg vol. cont. of the gases was as follows: N + rare gases = 95–8%. —*CA*

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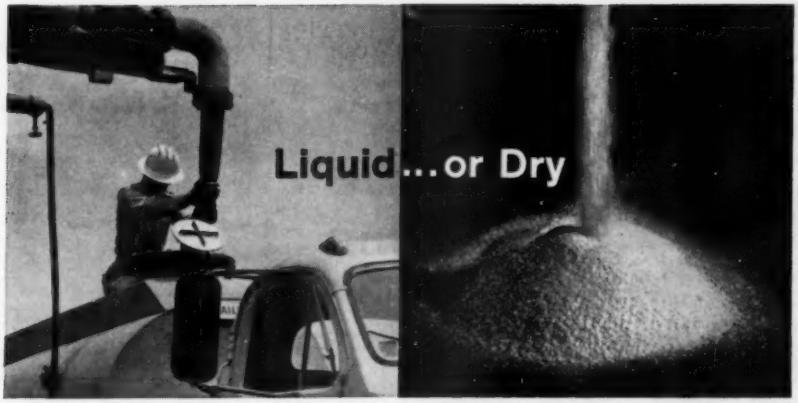

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Mueller Co.
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Buffalo Meter Co.
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Gamon Meter Div., Worthington Corp.
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Neptune Meter Co.
Rockwell Mfg. Co.

Meters, Filtration Plant, Pumping Station, Transmission Line:
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B-I-F Industries, Inc.—Builders
Fischer & Porter Co.
Simplex Valve & Meter Co.

Meters, Industrial, Commercial:
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Bailey Meter Co.
B-I-F Industries, Inc.—Builders
Buffalo Meter Co.
Calmet Meter Div., Worthington Corp.
Fischer & Porter Co.
Gamon Meter Div., Worthington Corp.
Hersey-Sparling Meter Co.
Neptune Meter Co.
Rockwell Mfg. Co.
Simplex Valve & Meter Co.

Mixing Equipment:
General Filter Co.
F. B. Leopold Co.

Motors, Electric:
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Marathon Electric Mfg. Corp.
Worthington Corp.

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Koppers Co., Inc.
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Pipe, Asbestos-Cement:
Atlas Asbestos Co. Ltd.
Johns-Manville Corp.
Keasbey & Mattison Co.

Pipe, Brass:
Anaconda American Brass Co.

Pipe, Cast Iron (and Fittings):
Alabama Pipe Co.
American Cast Iron Pipe Co.
James B. Clow & Sons
Trinity Valley Iron & Steel Co.
United States Pipe & Foundry Co.
R. D. Wood Co.

Pipe, Cement Lined:
American Cast Iron Pipe Co.
James B. Clow & Sons
Southern Pipe Div. of U.S. Industries
United States Pipe & Foundry Co.
R. D. Wood Co.

Pipe, Concrete:
American Pipe & Construction Co.
Lock Joint Pipe Co.
Vulcan Materials Co.

Pipe, Copper:
Anaconda American Brass Co.

Pipe, Plastic:
Keasby & Mattison Co.
Orangeburg Mfg. Co., Div. of The Flintkote Co.

Pipe, Steel:
Armco Drainage & Metal Products, Inc.
Bethlehem Steel Co.
Southern Pipe Div. of U.S. Industries

Pipe Cleaning Services:
Ace Pipe Cleaning, Inc.
Centriline Corp.
National Power Rodding Corp.
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Pipe Coatings and Linings:
American Cast Iron Pipe Co.
Centriline Corp.
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Koppers Co., Inc.
Pipe Linings, Inc.
Plastics & Coal Chemicals Div., Allied Chemical Corp.
Reilly Tar & Chemical Corp.
Southern Pipe Div. of U.S. Industries

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Ellis & Ford Mfg. Co.
Pilot Mfg. Co.
Jos. G. Pollard Co., Inc.
A. P. Smith Mfg. Co.
Wachs, E. H., Co.
Wheeler Mfg. Corp.

Pipe Jointing Materials; see Jointing Materials

Pipe Locators; see Locators, Pipe

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Jos. G. Pollard Co., Inc.
A. P. Smith Mfg. Co.

Potassium Permanganate:
Carus Chemical Co.

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Golden-Anderson Valve Specialty Co.
Mueller Co.
Ross Valve Mfg. Co.

Pumps, Boiler Feed:
DeLaval Steam Turbine Co.

Pumps, Centrifugal:
Allis-Chalmers Mfg. Co.
American Well Works
DeLaval Steam Turbine Co.
Peerless Pump Div.
C. H. Wheeler Mfg. Co.

Pumps, Chemical Feed:
B-I-F Industries, Inc.—Proportioneers
Fischer & Porter Co.
Wallace & Tiernan Inc.

Pumps, Deep Well:
American Well Works
Fiese & Firstenberger
Layne & Bowler, Inc.
Peerless Pump Div.

Pumps, Diaphragm:
Dorr-Oliver Inc.
Wallace & Tiernan Inc.

Pumps, Hydrant:
W. S. Darley & Co.
Jos. G. Pollard Co., Inc.

Pumps, Hydraulic Booster:
Peerless Pump Div.
Ross Valve Mfg. Co.

Pumps, Sewage:
Allis-Chalmers Mfg. Co.
DeLaval Steam Turbine Co.
Peerless Pump Div.
C. H. Wheeler Mfg. Co.

Pumps, Sump:
DeLaval Steam Turbine Co.
Peerless Pump Div.
C. H. Wheeler Mfg. Co.

Pumps, Turbine:
Layne & Bowler, Inc.
Peerless Pump Div.

Recorders, Gas Density, CO_2, NH_3, SO_2, etc.:
Fischer & Porter Co.
Permutit Co.
Wallace & Tiernan Inc.

Recording Instruments:
Bailey Meter Co.
B-I-F Industries, Inc.—Builders
Fischer & Porter Co.
Simplex Valve & Meter Co.
Wallace & Tiernan Inc.

Reservoirs, Steel:
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
Graver Tank & Mfg. Co.
Pittsburgh-Des Moines Steel Co.

Sand Expansion Gages; see Gages

Sleeves: see Clamps

Sleeves and Valves, Tapping:
James B. Clow & Sons
M & H Valve & Fittings Co.
Mueller Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.

Sludge Blanket Equipment:
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Met-Pro, Inc.
Permutit Co.

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Nalco Chemical Co.

Sodium Chloride:
International Salt Co.

Sodium Fluoride:
American Agricultural Chemical Co.
General Chemical Div., Allied Chemical Corp.

Sodium Hexametaphosphate:
Calgon Co.

Sodium Hypochlorite:
Wallace & Tiernan Inc.

Sodium Silicate:
General Chemical Div., Allied Chemical Corp.
Philadelphia Quartz Co.

Sodium Silicofluoride:
American Agricultural Chemical Co.
General Chemical Div., Allied Chemical Corp.
Tennessee Corp.

Softeners:
Dorr-Oliver Inc.
General Filter Co.
Hungerford & Terry, Inc.
Permutit Co.
Walker Process Equipment, Inc.

Softening Chemicals and Compounds:
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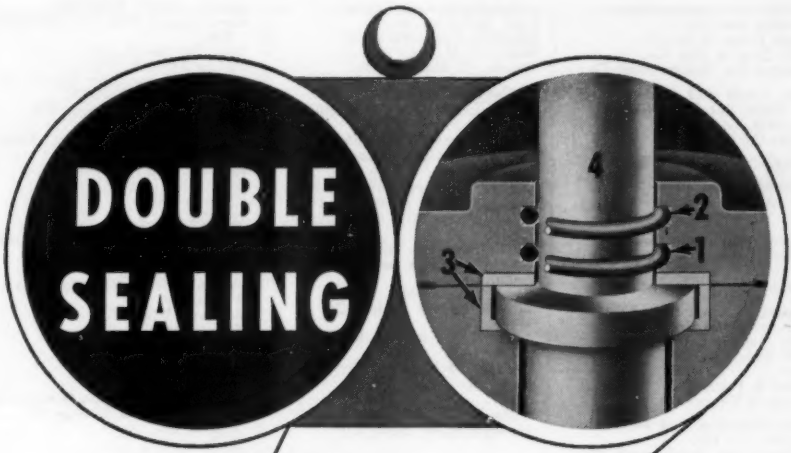
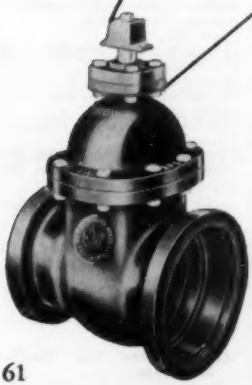
General Filter Co.
International Salt Co.
Nalco Chemical Co.
Permutit Co.
Tennessee Corp.

Standpipes, Steel:
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
Graver Tank & Mfg. Co.
Pittsburgh-Des Moines Steel Co.



SMITH VALVES WITH "O" RING SEALS



"O" Rings, due to their simplicity and reliability have been used for many years in military and industrial products. Smith non-rising stem valves are available with *double* "O" Ring seals. The lower "O" Ring (1) seals the internal pressure, while the upper "O" Ring (2) is a combined external dirt and reserve pressure seal. This construction eliminates packing, gland adjustment, and conventional stuffing box. All Smith AWWA specification valves are equipped with stem (thrust) collar bushings (3) and extra large, high strength bronze stems (4). These and other well known Smith superior design features assure positive operation, long service life, and minimum maintenance.

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THE A.P. SMITH MFG. CO.

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Chicago Bridge & Iron Co.
Graver Tank & Mfg. Co.
Pittsburgh-Des Moines Steel Co.

Stops, Curb and Corporation:
Ford Meter Box Co.
Hays Mfg. Co.
Mueller Co.

Storage Tanks: see Tanks

Strainers, Suction:
James B. Clow & Sons
R. D. Wood Co.

Surface Wash Equipment:
Golden-Anderson Valve Specialty Co.
Permutit Co.

Swimming Pool Sterilization:
B-I-F Industries, Inc.—Builders
B-I-F Industries, Inc.—Omega
B-I-F Industries, Inc.—Proportioneers
Fischer & Porter Co.
Wallace & Tiernan Inc.

Tank Painting and Repair:
Koppers Co., Inc.
National Tank Maintenance Corp.

Tanks, Prestressed Concrete:
Preload Co., Inc.

Tanks, Steel:
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Chicago Bridge & Iron Co.
Graver Tank & Mfg. Co.
Pittsburgh-Des Moines Steel Co.

Tapping-Drilling Machines:
Hays Mfg. Co.
Mueller Co.
A. P. Smith Mfg. Co.

Tapping Machines, Corp.:
Hays Mfg. Co.
Mueller Co.

Taste and Odor Removal:
B-I-F Industries, Inc.—Builders
B-I-F Industries, Inc.—Proportioneers
General Filter Co.
Industrial Chemical Sales Div.
Met-Pro, Inc.
Permutit Co.
Wallace & Tiernan Inc.

Turbidimetric Apparatus (For Turbidity and Sulfate Determinations):
Wallace & Tiernan Inc.

Turbines, Steam:
Allis-Chalmers Mfg. Co.
DeLaval Steam Turbine Co.

Valve Boxes:
James B. Clow & Sons
Ford Meter Box Co.
M & H Valve & Fittings Co.
Mueller Co.
Rockwell Mfg. Co.
A. P. Smith Mfg. Co.
Trinity Valley Iron & Steel Co.
R. D. Wood Co.

Valve-Inserting Machines:
Mueller Co.
A. P. Smith Mfg. Co.

Valve-Operating Units:
B-I-F Industries, Inc.
Filtration Equipment Corp.
Wachs, E. H., Co.
Wheeler, C. H., Mfg. Co.

Valves, Altitude:
Allis-Chalmers Mfg. Co., Hydraulic Div.
Golden-Anderson Valve Specialty Co.
Ross Valve Mfg. Co., Inc.

Valves, Butterfly, Check, Flap, Foot, Hose, Mud and Plug:
Allis-Chalmers Mfg. Co., Hydraulic Div.
B-I-F Industries, Inc.—Builders
James B. Clow & Sons
DeZurik Corp.
Kennedy Valve Mfg. Co.
M & H Valve & Fittings Co.
Mueller Co.
Pelton Div., Baldwin-Lima-Hamilton
Rockwell Mfg. Co.
R. D. Wood Co.

Valves, Detector Check:
Hersey-Sparling Meter Co.

Valves, Electrically Operated:
Allis-Chalmers Mfg. Co., Hydraulic Div.
B-I-F Industries, Inc.—Builders
James B. Clow & Sons
Darling Valve & Mfg. Co.
Golden-Anderson Valve Specialty Co.
Kennedy Valve Mfg. Co.
M & H Valve & Fittings Co.
Mueller Co.
Rockwell Mfg. Co.
A. P. Smith Mfg. Co.

Valves, Float:
James B. Clow & Sons
Golden-Anderson Valve Specialty Co.
Rockwell Mfg. Co.
Ross Valve Mfg. Co., Inc.

Valves, Gate:
James B. Clow & Sons
Darling Valve & Mfg. Co.
Dresser Mfg. Div.
Kennedy Valve Mfg. Co.
M & H Valve & Fittings Co.
Mueller Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Valves, Hydraulically Operated:
Allis-Chalmers Mfg. Co., Hydraulic Div.
B-I-F Industries, Inc.—Builders
James B. Clow & Sons
Darling Valve & Mfg. Co.
DeZurik Corp.
Golden-Anderson Valve Specialty Co.
Kennedy Valve Mfg. Co.
F. B. Leopold Co.
M & H Valve & Fittings Co.
Mueller Co.
Pelton Div., Baldwin-Lima-Hamilton
Rockwell Mfg. Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Valves, Large Diameter:
Allis-Chalmers Mfg. Co., Hydraulic Div.
James B. Clow & Sons
Darling Valve & Mfg. Co.
Golden-Anderson Valve Specialty Co
Kennedy Valve Mfg. Co.
M & H Valve & Fittings Co.
Mueller Co.
Pelton Div., Baldwin-Lima-Hamilton
Rockwell Mfg. Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Valves, Regulating:
Allis-Chalmers Mfg. Co., Hydraulic Div.
DeZurik Corp.
Golden-Anderson Valve Specialty Co.
Mueller Co.
Rockwell Mfg. Co.
Ross Valve Mfg. Co.

Valves, Swing Check:
James B. Clow & Sons
Darling Valve & Mfg. Co.
Golden-Anderson Valve Specialty Co
M & H Valve & Fittings Co.
Mueller Co.
Rockwell Mfg. Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Venturi Tubes:
B-I-F Industries, Inc.—Builders
Rockwell Mfg. Co.
Simplex Valve & Meter Co.

Waterproofing:
Inertol Co., Inc.
Koppers Co., Inc.
Plastics & Coal Chemicals Div., Allied Chemical Corp.

Water Softening Plants; see Softeners

Water Supply Contractors:
Layne & Bowler, Inc.

Water Testing Apparatus:
LaMotte Chem. Products Co.
Wallace & Tiernan Inc.

Water Treatment Plants:
American Well Works
Chain Belt Co.
Chicago Bridge & Iron Co.
Dorr-Oliver Inc.
General Filter Co.
Hungerford & Terry, Inc.
Infilco Inc.
Met-Pro, Inc.
Permutit Co.
Pittsburgh-Des Moines Steel Co.
Walker Process Equipment, Inc.
Wallace & Tiernan Inc.

Well Drilling Contractors:
Layne & Bowler, Inc.

Well Reconditioning and Formation Testing:
Halliburton Co.
Layne & Bowler, Inc.

Wrenches, Ratchet:
Dresser Mfg. Div.

Zeolite: see Ion Exchange Materials

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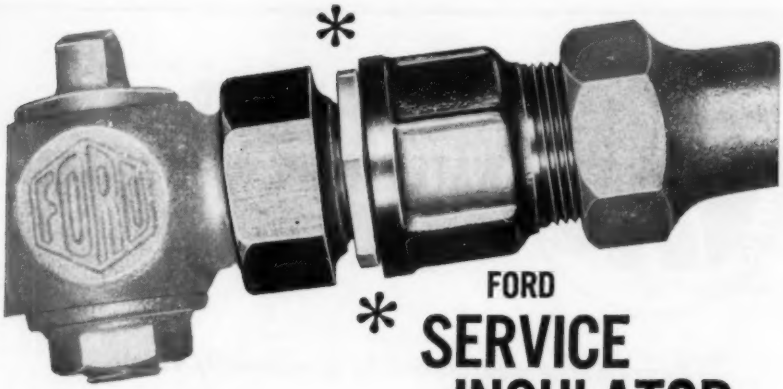


FORD SERVICE INSULATOR

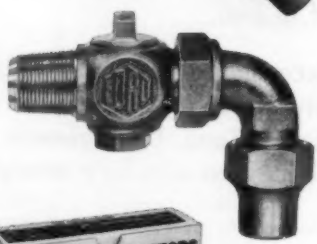
This simple insulating unit, which includes a nylon dielectric bushing, can be attached to the outlet of a standard corporation stop. Connects to ordinary copper tubing with standard nut. Provides positive service insulation without gaskets or loose parts. Requires no special tools.

Always Specify

FORD CORPORATION STOPS..

made of finest water works brass.

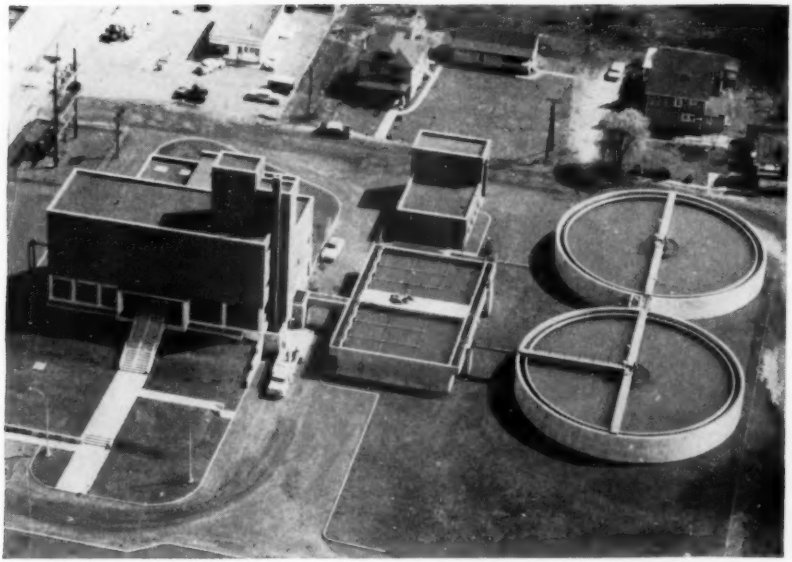
Precision machined to accurate dimensions. Generous weight with good finish. Thick - substantial operating head. Ample wrench flat with sharp corners. Straight - Eighth - or Quarter bends available in ¾" and 1" sizes.



This attractive, strong Ford carton holds five ¾" or four 1" size Stops for easier handling, simpler storage and better protection from dirt and damage.

FORD for *better* water services

THE FORD METER BOX COMPANY, INC., Wabash, Indiana



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Treats Ohio River Water in Dorr-Oliver Equipment

A quarter-century ago, the City of Ashland, Kentucky, installed a new water treatment plant which incorporated Dorrco® Flocculators. Today, removed from the earlier plant's flood-ridden area, an entirely new plant has been constructed, again incorporating Dorr-Oliver treatment units.

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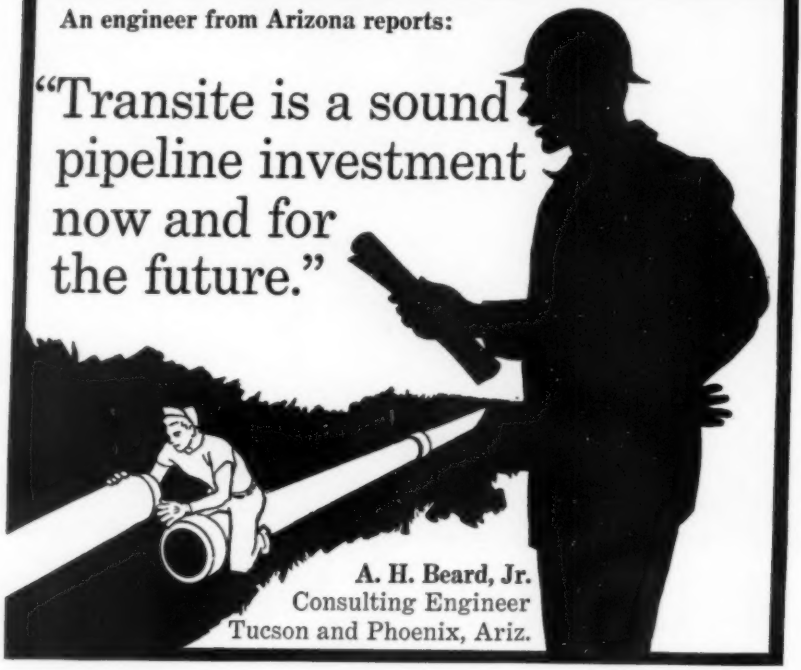

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"Experience has shown that Johns-Manville Transite® Pipe offers the value we seek. It has proved reliable and at very favorable costs. In addition to savings in handling and installing, Transite's unique Ring-Tite® joints assure that leakage tests will not upset job costs. In numerous tests, Transite lines have regularly been under the 'allowable' specification.

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